EGYPTIAN GOVERNMENT

MINISTRY OF PUBLIC WORKS

REPORT

ON

THE ASWAN HYDRO-ELECTRIC PROJECT

March 1950

Vattenbyggnadsbyrån (VBB)
Consulting Engineers and Architects
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March 31st, 1950

Messrs. Kennedy & Donkin,
Consulting Engineers,
12, Caxton Street,
London, S.W.1.

Sirs,

In accordance with your request we have studied the Tender Documents as well as your Report and Memos on the Aswan Hydro-Electric Project submitted to us and have also made certain special investigations. We have the honour to submit our Report herewith, and at the same time to traverse its salient features.

The Aswan Dam

The Aswan Dam is a magnificent structure built of Aswan granite, one of the best building materials in the world. During the visit to the dam of our representatives it was found that the masonry, as far as could be judged by ocular inspection, was in good condition. It is, however, a reasonable precautionary measure to investigate, at certain intervals of years, the quality of the masonry, or concrete, in any dam. If the masonry has not recently been investigated, we recommend that cores of the masonry be obtained by diamond drilling for examination and testing.

It would be of great importance for the power generation at Aswan that the low water level in the reservoir could be fixed at R.L. 101.0 instead of at R.L. 98.0. We understand that the second heightening was designed for a storage level of R.L. 122.0. The reduction of storage, corresponding to the above-mentioned raising of the low water level, could be made

good by raising the upper storage level by about 0.55 m, the amount of storage thus gained being the same as the storage lost between R.L. 101.0 and R.L. 98.0. Should such a raising of the storage level not be considered advisable, the deficiency could be made good by creating further storage at some other place, e.g. at Wadi Halfa.

The extreme high water level downstream of the power plant is of great importance for the design of the substructures. It is recommended that the floods in the Nile be further investigated, and that the extreme floods which occurred in 1817 and in 1818 be taken into account.

The Tender Scheme

The Tender Scheme was, we understand, prepared in 1932 by the Mechanical and Electrical Department of the Ministry of Public Works, and was elaborated under certain severe restrictions with reference to various conditions, such as those relating to the distance from the dam to the power house, to the non-interference with the existing dam, to the excavation to be carried out without using explosives, etc.

The Tender Scheme, as later on elaborated by the Consultants, and also the modification thereof, are marked by the same restrictions.

Three modifications of the Tender Scheme have been prepared by the Consultants, viz.:-

Scheme "A", an indoor station,
Scheme "B", an outdoor station, and
Scheme "C", a semi-outdoor station.

In summer Aswan has an extremely hot climate with a blazing sun, and even at moderate winds the atmosphere carries a certain amount of dust and grit. Therefore, during certain times of the year it would meet with considerable difficulties to dismantle and re-erect a turbine or an alternator without a superstructure. For the above reason we do not feel justified in recommending Scheme "B" or Scheme "C" to be carried out.

A special study has been made of the design of the power house, and recommendations are given regarding steps to be taken to reduce the cost.

In Scheme "A" the modifications suggested by the Consultants would result in a saving of £E 993,000, as compared with the Tender Scheme. By introducing such further modifications as suggested by us there would be an additional saving of about £E 650,000.

The arrangement proposed for leading the water from the reservoir to the power house, which remains unchanged in all the above schemes, is, in our opinion, not satisfactory. It entails the introduction of surge tanks, and the cost of maintenance of these tanks and of the 90 steel penstocks would be high.

The plant has been subject to extensive model tests carried out by Professor E. Meyer-Peter. As a result of these tests Professor Meyer-Peter recommends that the water level in the reservoir upstream of the dam should never be drawn down below R.L. 100.0 as an absolute minimum. We suggest R.L. 101.0 as the lowest water level in the reservoir. Should the low water be drawn down below this level, the operation of the plant might have to be suspended.

For these reasons and for other reasons, given in the Report, neither the original Tender Scheme, nor the Modified Scheme "A", can, in our opinion, be recommended.

Alternatives

It might be considered, as an alternative to the Tender Scheme, to move the power house closer to the dam, to enlarge the bottom outlets, and to lead the water in short concrete conduits at a much lower velocity than in the Tender Scheme, whereby surge tanks could be omitted. The work of enlarging the bottom outlets would, however, be difficult and might involve certain risks. For this and other reasons this alternative has not been further investigated. Instead two other possible alternatives are being dealt with in the Report under the headings of

The Forebay Scheme, and
The Tunnel Scheme.

In the Forebay Scheme the power house is located at the same place as in the Tender Scheme. The pipe lines and the surge tanks are omitted and, instead, the water is con-

veyed to the turbines by means of a forebay created by an intake dam close to the power house, and by two wing walls. Two alternative positions of the west wing wall have been considered.

In the Tunnel Scheme all the works for the power production are moved away and located at a safe distance from the dam. The power house is placed about 500 m upstream of the dam. The tail race tunnels are located deep into the rock and discharge the water downstream of the dam at about the same place at which it would be discharged according to the Tender Scheme.

The Forebay Scheme

In the Forebay Scheme part of the existing dam would be submerged in water. Thus, this part of the dam would be relieved of water pressure with the exception of that corresponding to the loss of head, which is small. This part of the dam would then have the same temperature on the upstream as on the downstream side. The existing dam east of the eastern wing wall and west of the western wing wall would, as at present, have water pressure on the upstream side, only, and be subject to variations of temperature due to water on the upstream side and to air on the downstream side. To relieve the stresses it is suggested that permanent expansion joints be arranged in the existing dam at the junctions between the wing walls and the dam. The expansion joints would be arranged so as to allow of movements of the structures independently of each other.

The Tunnel Scheme

The Tunnel Scheme consists of an intake and a power house located on the western bank about 500 m upstream of the existing dam. Water would be conveyed to the intake and to the power house by a short head race. In the power house the water would be led to the turbines in spiral casings of reinforced concrete. This would be more economical and render possible a shorter time of construction than the use of steel spiral casings. A vast depression in the ground would be utilized for the construction of a surge basin immediately downstream of the power house. Four tunnels would be provided,

excavated in solid rock. The average length of the tunnels would be about 820 m, and the tunnels would be located so deep in the rock that the water would be discharged underneath Locks Nos. 2, 3, and 4.

Arrangements are provided so that the diversion of water from the river into the head race would not cause any difficulty for the traffic to pass into or out of the navigation channel.

Comparison of Schemes

For comparing the alternative schemes not only the capital cost but also the annual cost of depreciation and maintenance should be taken into account as well as the amount of power obtained, the time of construction and certain other features. It appears practical, for the purpose of comparison, to build up "Comparative Figures" of cost by adding together, for the different schemes, their capital cost and the capitalized value of the annual cost.

The figures referred to are given in the table below.

	Capital cost	Capital value of depreciation and maintenance (4 per cent interest)	Comparative Figure
	£E	£E	£E
Modified Scheme "A"	7,930,000	3,250,000	11,180,000
Forebay Scheme:			
Alternative 1	7,500,000	2,200,000	9,700,000
Alternative 2	7,610,000	2,200,000	9,810,000
Tunnel Scheme	8,310,000	1,925,000	10,235,000

As far as the comparative figures are concerned the Modified Scheme "A" is the least economical. The improvement in economy by adopting the Tunnel Scheme corresponds to about £E 1 million. On the other hand, the difference between the Comparative Figure of the Tunnel Scheme and those of the two alternatives of the Forebay Scheme is about £E 0.5 million, only.

The loss of head and of power at a gross head of 11.5 m would be as follows:-

	Modified Scheme "A"	Forebay Scheme	Tunnel Scheme
Gross head, m	11.50	11.50	11.50
Loss of head, m	1.47	0.37	1.29
Net head, m	10.03	11.13	10.11
Discharge, m^3/s	1313	1383	1324
Turbine efficiency, per cent	80	81	80
Output, kW	96,800	114,600	98,400

For the purpose of this Report, the output calculated above may be considered as firm power. As will be seen, the gain in firm power by adopting the Forebay Scheme instead of the Tender Scheme would be no less than 17,800 kW. The gain in firm power by adopting the Tunnel Scheme instead of the Tender Scheme would be 1,600 kW. As mentioned below, however, by adopting the Tunnel Scheme there would be a possibility of obtaining considerably more firm power in the future than in any of the other schemes, by installing four, perhaps five, low head units.

On account of air being sucked into the turbines at low water in the reservoir, the loss of firm power in the Modified Scheme "A" may be considerably greater than that given above.

It is estimated that the Forebay Scheme can be executed in about the same time as the Modified Scheme "A".

With reference to the Tunnel Scheme, which is based upon the use of spiral casings of reinforced concrete, it is estimated that the first unit would be put into operation three months later than the first unit in the Modified Scheme "A" or in the Forebay Scheme, whereas the last unit will be put into operation at the same time as in the other schemes.

A comparison of the three schemes should be made on a broad basis. It should be kept in mind that the total cost of the hydro-electric project will be well above £E 15 millions. It goes without saying that, before such a vast project is embarked upon, there must be no doubt whatsoever about the suitability and efficiency of the plant.

As mentioned above, for technical reasons the Tender Scheme cannot, in our opinion, be recommended. Besides, the

above comparison of costs has shown that it would be less economical than the alternatives considered, due to its high cost of depreciation and maintenance and to its smaller output.

The Forebay Scheme is based on recognized and common practice for utilizing water power. The design put forward for the junction between the wing walls and the existing structure should satisfy all reasonable requirements. It would, however, be necessary to make rock excavation for the wing walls near the existing dam. Provided that this excavation be carried out with great care, without using explosives, and that the rock be grouted by new, recently developed methods, the excavation need not be so deep as would have been the case some years ago. There should, therefore, be no objection to the Forebay Scheme being accepted.

Also the Tunnel Scheme is based on recognized and common practice for utilizing water power. In this scheme works near the existing dam would be avoided. According to the estimate of cost this scheme would be somewhat more expensive than the Forebay Scheme.

An attractive feature of the Tunnel Scheme is that it would be possible in the future to install a number of low head units downstream of the dam, utilizing the present excavation, whereby a greater amount of firm power would be obtained than in any of the other schemes. If no change can be made of the present rules of drawing down the reservoir, this feature is of paramount importance, since the economy of the fertilizer plant and of the steel works will greatly depend upon the amount of firm power available.

In view of this, too much importance should not be attributed to the difference in capital cost between the Forebay Scheme and the Tunnel Scheme.

Recommendation for Immediate Action

Certain investigation work for the Tunnel Scheme would be required. The main object thereof would be to determine the elevation of the contact zone between granite and sandstone in the area of the proposed tunnels. Two complete equipments of diamond drills are available and can be shipped to Aswan immediately on receipt of order. An offer for this

is attached to the Report. It is estimated that the necessary drilling would take about two months after the order has been placed. A contour survey should also be made of the area to be occupied by the head race, intake, power house, and surge basin of the Tunnel Scheme.

It is recommended that steps be taken immediately to carry out the above-mentioned investigations. This should, in our opinion, be done irrespectively of any other steps which may be taken in regard to the Hydro-Electric Project.

May we, at this juncture, take the opportunity of acknowledging the most helpful and friendly co-operation which has taken place between your firm and your Associates, and ourselves.

Yours faithfully,
For VATTENBYGGNADSBYRÅN (VBB)

B. Hellström

B. Hellström

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REPORT
ON
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1. INTRODUCTORY

The Aswan Dam is situated on the Nile about 700 kilometres south-south-east of Cairo and about 200 kilometres south of Luxor, as the crow flies. It has hitherto served for irrigation purposes only. The dam has a length of about 2,000 metres and impounds over 5,000 million cubic metres of water between the storage level at R.L. 121.0 and the low water level at R.L. 98.0, to which level the water is drawn down for supplying water for irrigation purposes during the summer season.

For many years it has been proposed to use water released from the Aswan reservoir for generation of hydro-electric power, and many projects have been considered for realizing this purpose. This Report deals with the latest of these projects, here referred to as the Tender Scheme, and also with two alternatives, referred to as the Forebay Scheme and the Tunnel Scheme.

Terms of Reference

The Consulting Engineers for the Aswan Hydro-Electric Project, in this Report referred to as the Consultants, are:-

Messrs. Kennedy & Donkin, London,
Messrs. Binnie, Deacon & Gourley, London,
Messrs. Scott & Wilson, London, and
Gebrüder Gruner Ingenieurbureau, Basel.

The Consulting Architect is Sir Giles Gilbert Scott, O.M., R.A., London.

In October, 1949, Vattenbyggnadsbyrån (VBB), Consulting Engineers and Architects, Stockholm, were asked by Messrs. Kennedy & Donkin to give an opinion on the methods of reducing the cost of the civil engineering works contemplated for the project.

The terms of reference finally agreed upon are given in a letter from Messrs. Kennedy & Donkin, dated November 1st, 1949, as follows:-

> "To comment on the report of Messrs. Kennedy &
> "Donkin on the steps which can be taken to reduce
> "the cost of the civil and mechanical works of the
> "Aswan Hydro-Electric Project and to suggest any
> "other possible steps which could be taken."

Later on the Consultants submitted a report dated November, 1949, on possible reductions in capital expenditure for the civil engineering and building works.

In order to study local conditions Professor B. Hellström, of VBB, visited Egypt between December 21st, 1949, and February 8th, 1950. Mr. Nils Berg, C.E., of VBB, also visited Egypt between January 23rd and February 8th, 1950.

Acknowledgements

VBB desire to express their appreciation of the excellent facilities provided by, and the hearty co-operation of H.E. Osman Moharram Pasha, Minister of Public Works. They are also greatly indebted to Mustafa Fathy Bey, President of the Power Commission, and to Murad Fahmy Bey, President of the Hydro-Electric Power Department, for their advice and assistance.

The representatives of VBB, who visited Egypt, wish gratefully to acknowledge their valuable collaboration with Mohammed Amin Fathy Bey, Assistant Undersecretary of State and Vice President of the Hydro-Electric Power Department, Dr. Eng. Hassan Zaky Bey, Chief Inspector of Irrigation for Upper Egypt, Mohammed Ahmed Eteika Bey, Resident Engineer (Civil), Aswan Hydro-Electric Scheme, Mr. Abdel Moety Amer, Director of Technical Research, Hydro-Electric Power Department, and Mr. Taha Zaky, Secretary, Hydro-Electric Power Department.

Mr. W. Borgquist, formerly Director-General, and Mr. H. Munding, formerly Chief Mechanical Engineer, both of the State Power Board, Sweden, have rendered valuable assistance for which VBB wish to express their gratitude.

Acknowledgements are also due to the many representatives of the Consultants with whom a most useful and friendly collaboration has taken place.

2. THE ASWAN DAM AND RESERVOIR

The Dam .

The Aswan Dam is a magnificent structure built of Aswan granite, one of the best building materials in the world. The original dam was constructed between 1898 and 1902. The road level was then at R.L. 109.0 and the storage level at R.L. 106.0, the contents of the reservoir being about 1,000 million cubic metres of water above R.L. 94.0.

After a few years of operation the original dam was heightened to R.L. 114.0, the storage level being raised to R.L. 113.0 and later on, gradually, to R.L. 113.9. Thanks to the heightening of the dam the storage was increased to about 2,500 million cubic metres at a storage level of R.L. 113.0.

The second heightening of the dam took place in 1929 to 1934, when the road level was raised to R.L. 123.0, and the storage level, by a Ministerial Order, dated November 20th, 1929, was fixed at R.L. 121.0. This corresponds to a capacity of over 5,000 cubic metres.

The water is now discharged through the dam by 179 bottom sluices, divided into 18 sets. Of these 140 sluices have a width of 2.0 m and a height of 7.0 m, and their sill levels are at R.L. 87.65 (65 sluices) and at R.L. 92.0 (75 sluices), respectively. The other 39 sluices are smaller and their sills are at higher levels.

On the west side of the dam there is a navigation canal with a flight of five locks at the dam and an additional lock further downstream.

During the inspection of the dam by the representatives of VBB it was found that the leakage through the dam was exceedingly small and that, as far as could be judged by ocular inspection, the masonry was in good condition. An inspection of one of the gate wells showed hardly any leakage through the upstream wall constructed at the first and second heightenings, whilst the wall of the gate well in the original dam was moist or wet.

It is a reasonable precautionary measure to investigate, at certain intervals of years, the quality of the masonry, or concrete, in any dam. Such periodical investigations are carried out practically at all large dams in the world. We are not aware whether the masonry of the dam has recently been investigated by core drilling. If not, we recommend that cores of the masonry be obtained by diamond drilling for examination and testing.

The Reservoir

According to information received, the water level above the dam will vary in accordance with drawing C.D. 9 of the contract drawings for the Tender Scheme. During January to April the water level will be kept at, or near, the storage level R.L. 121.0. About May the water level will gradually be lowered, the lowest level of R.L. 98.0 being reached before the flood, generally towards the end of July. During the flood period, in August to October, the water level will be kept at R.L. 105.0.[1] The filling of the reservoir will take place towards the end of the flood period, the storage level of R.L. 121.0 being reached at the end of November or the beginning of December.

It would be of paramount importance for the power generation at Aswan if the low water level in the reservoir could be fixed at R.L. 101.0 instead of at R.L. 98.0. According to a diagram of the contents of the reservoir, included in the album of drawings of the second heightening, this would entail a reduction of the storage volume of about 225 million cubic metres. We understand that the second heightening was designed for a storage level of R.L. 122.0. The above-mentioned reduction of the storage could be made good by raising the storage level by about 0.55 m, the amount of storage in the reservoir between R.L. 121.55 and R.L. 121.0 being about 225 million cubic metres. Should such a raising of the storage level not be considered advisable, the deficiency could be made good by creating further storage at some other place, e.g. at Wadi Halfa as mentioned later on in this Report.

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1) In drawing C.D. 9 the level given during floods is R.L. 103.0, but the rules have since been changed.

Flood Discharge and High Water Level

The water is discharged through the dam into the eastern, the central, and the western channels of the Nile downstream of the dam. Since the power house is proposed to be located in the western channel, it is of importance to determine the highest possible water level in this channel immediately downstream of the power house.

There are two gauges on the embankment of the navigation channel below the locks, the South Gauge, erected in 1937, located about 800 m downstream of the dam, and the North Gauge, erected in 1942, 200 m further downstream. The highest water levels observed at these gauges are as follows:-

		Water Level	
		South Gauge	North Gauge
13.9.1938	R.L.	95.65	-
8.9.1943	R.L.	95.69	95.45
9.9.1946	R.L.	95.94	95.60

The downstream high water level at the power house on which the design of the project has been based is R.L. 97.0. This level has been obtained by adding to the figure of R.L. 95.94 the estimated friction losses in 800 m length of channel, as well as a certain margin of safety.

The Consultants have estimated that, on an average, once in a thousand years a high water level of R.L. 97.3, corresponding to a discharge of 14,500 cubic metres per second, would be reached or exceeded.

We are informed that the maximum discharge of the flood in 1946 was practically of the same magnitude as that of the highest flood on record for the last hundred years or so, but it is also understood that, during this period, there has not been any really high flood in the Nile. To judge from several accounts, however, extreme floods occurred in 1817 and in 1818. It is recommended that the floods in these years be investigated and taken into account, and a Note on these floods is attached, Appendix 1.

Further Storage

Mr. W. Borgquist has previously suggested to the Egyptian Government that the possibilities of obtaining further storage at Wadi Halfa be investigated. On account of what has been said above with reference to storage in the Aswan reservoir, we fully concur with this suggestion and need not stress the advantages for irrigation as well as for power generation of further storage. If and when such storage will be provided, it would be of great advantage to raise the low water level in the reservoir to, say, R.L. 105.0.

3. TENDER SCHEME

Description of Original Tender Scheme

The Tender Scheme is based upon the power house being located near the western bank downstream of the dam below the bottom outlets of Sets Nos. 2 to 6.

The power house is planned to contain seven large main units, each of a capacity of 65,000 H.P. (49,000 kW), and two house units, each of a capacity of 16,000 H.P. (12,500 kW), all these units to be operated at any reservoir level. In addition, the power house would contain two low head units which would be used at a water level in the reservoir of between R.L. 105.0 and R.L. 98.0. These latter units would be operated during low water conditions in the reservoir, only, and are intended for supplying electricity when the head is small and the main units cannot deliver full output, due to reduced head.

For the large main units mentioned above the existing bottom outlets in the dam would be used for conveying water through the dam by inserting in each outlet three steel penstocks placed on top of each other, each having a diameter of 1.8 m. The clearance between the penstocks and the lining of the sluices in their narrowest part would be about 0.1 m. It has not yet been decided how the penstocks would be fitted to the bottom outlets, and prospective tenderers have been asked to submit proposals on this matter. In any case this work is apt to be difficult and intricate.

The penstocks issuing from four bottom outlets, i.e. 12 pipes for each main unit, are to be assembled to a manifold. The penstocks would have a diameter of 2.25 m between the dam and the manifold. From the manifold a single steel-lined rectangular concrete conduit, gradually changing to a circular form, and continued by a steel pipe, 7.8 m in diameter, would lead the water to the turbine spiral casing. Close to the manifold the conduit would be connected to a surge tank having a diameter of 26 m and a top elevation of R.L. 125.0.

The centre line of the units would be located at a distance of 120 m from the axis of the dam and the power house would accomodate, in its bottom part, the turbines enclosed in steel spiral casings, and, in its upper part, the generators and travelling cranes.

To each one of the house units water would be conveyed, from one sluice opening, only, by three penstocks of the same size as the penstocks leading to the main units and similarly placed on top of each other.

The two low head units and one spillway would be located in the centre of the power house. The arrangements for conveying water to the low head units would consist of a basin with two wing walls, having their crest level at R.L. 107.0. The rock excavation for the cut-off trench of the wing walls is suggested to be carried down to about R.L. 81.50, i.e. to a depth of 8.3 m below the surface of the present apron downstream of the dam. We are informed that the wing walls are proposed to be concreted against the existing dam and buttresses. This work, however, has not yet been approved by the Government and the installation of the low head units and the construction of the low head basin has therefore been deferred.

Contract documents for obtaining tenders for the civil engineering works of the Tender Scheme were sent out in March, 1949. Tenders were to be submitted before January 11th, 1950, but in consideration of a new development of the matter it was decided that the date of opening the tenders should be postponed.

Contract documents for the penstocks and surge tanks were sent out in November, 1949, and the tenders are to be opened on May 5th, 1950.

Consultants' Report of November, 1949

In the report of November, 1949, on possible reductions in capital expenditure the Consultants have considered three alternative schemes, referred to as follows:-

Indoor Station, Scheme "A", Drawing EG. 288
Outdoor Station, Scheme "B", Drawings EG. 281/282
Semi-outdoor Station, Scheme "C", Drawings EG. 283 and 287.

Scheme "A" is a modification of the Tender Scheme.

In Scheme "B" the turbine house is of the open type without any superstructure.

Also in Scheme "C" the turbine house is of the open type but for operational convenience, a covered accommodation along the units is provided for the full length of the station.

Power stations of type "B" have occassionally been used in various parts of the world (U.S.A., Sweden, Finland), and the type employed in Scheme "C" has also been used (inter alia for a medium size power station in Sweden).

In summer Aswan has an extremely hot climate with a blazing sun, and even during moderate winds the atmosphere carries a certain amount of dust and grit. Therefore, during certain times of the year it would meet with considerable difficulties to dismantle and re-erect a turbine or an alternator without a superstructure.

For the above reason we do not feel justified in recommending Scheme "B" or Scheme "C" to be carried out, and in this Report these two schemes will not be dealt with any further.

Superstructure for Scheme "A"

In the Tender Scheme the power house is high and wide; the substructure contains large quantities of reinforced concrete and the superstructure a great amount of steel.

In the Tender Scheme as well as in Scheme "A" the height of the crane rail above turbine room floor is 17.0 m. This height is unusually great. Owing to the provision of an auxiliary crane to run below the main cranes and to be used during erection of the steel spiral casings, the above-mentioned height can, however, only be reduced to 16.0 m.

The inside width of the power house in the Tender Scheme is 26.4 m, and this width has been maintained in Scheme "A". The span of the cranes in both cases is 22.2 m. Should such a power plant be built in Sweden and orders for all the turbines placed with the same manufacturer, the span of the crane would be about 15.3 m. In a letter to Messrs. Kennedy & Donkin of February 17th, 1950, Messrs. Escher Wyss Ltd. (EW) state that a 15.3 m span can be accepted but must be considered as a minimum and suggest that, if possible, the span be increased to 15.5 or 16.0 m. Karlstads Mekaniska Werkstad (KMW) concur with these views.

At Aswan, however, there are different manufacturers of the turbines, using different arrangements for the auxiliary machinery. For the KMW units the distance required between the

centre of the turbines and the upstream crane rail is 8.25 m, whilst EW require 9.0 m from the centre of the turbines to the downstream crane rail, making a total span of 17.25 m. This span is sufficient for passing heavy parts of machinery over and above units already erected. Such handling of heavy machinery is common practice in many countries. Therefore there should be no objection to such an arrangement and to a span limited to 17.25 m.

We understand, however, that the Consultants prefer the machinery to be passed between the erected units and the downstream wall of the power house. For this purpose a span of 19.0 m of the crane would be required.

In the Tender Scheme and in Scheme "A" the distance between the main units is 30 m or thereabout. This distance is required on account of the spiral casings being made of steel but is also due to the spacing of the bottom sluices in the dam.

A considerable saving of cost would be effected by substituting a reinforced concrete superstructure for steel, as shown on Plate 1.[1] In this design the crane girders are of structural steel. These may, however, suitably be replaced by reinforced concrete arches as suggested later on by the Consultants. To prevent the rays of the sun to enter the power house it is suggested that the outer roof should protrude outside of the façades.

Substructure of Scheme "A"

In the Tender Scheme as well as in Scheme "A" the steel spiral casings are enclosed in heavy concrete structures. These structures are designed to withstand, during construction, full water pressure from the outside at high water level (R.L. 97.0) downstream of the power house, before the floors at R.L. 92.8 and R.L. 98.5 have been constructed. We understand that the structures are also designed to withstand full water pressure from the inside corresponding to the maximum elevation of the reservoir water level (R.L. 121.0) in case any one of the

1) In this drawing, which was prepared in December, 1949, for the sake of discussion, the span of the travelling crane is given at 15.3 m and the height of the crane rail above floor is 14.0 m which was the height required before the auxiliary crane was suggested.

spiral casings should burst. Should this assumption not be correct, provision should be made to cope with such an emergency, since experience proves that the possibility not only of a penstock but even of a steel spiral casing bursting is not as remote as generally assumed.

Two suggestions for reducing the cost are worth considering, viz. :-

(a) To delete the upstream wall of the substructure so as to let the water escape into the penstock area between the power house and the dam in case a steel spiral casing should burst, and to arrange outlets from this area to avoid too high water pressure on the other steel spiral casings and penstocks, in case they should be empty.

(b) To substitute spiral casings of reinforced concrete for steel spiral casings.

With reference to (a) an inexpensive design would consist of concrete buttresses between the units, with arches spanning between the buttresses, as shown on Plate 1. The arches will have to withstand the water pressure from the downstream side at times of flood (R.L. 97.0) and also tension corresponding to the difference between the water levels in the penstock area and in the tail race, should the penstock area become flooded.

To provide an escape for the water which, in the case of an accident either to a spiral casing or to a penstock, would accumulate in the penstock area, outlets could be arranged in the side walls enclosing this area and/or under the arches, as shown on Plate 1. In the latter case the gates would be of the flapper type and open up towards the downstream side under a super-pressure from the upstream side.

With reference to (b) it should be mentioned that a spiral casing of reinforced concrete, if not executed with great care, may show minor defects, such as leakage through inferior concrete or small cracks, which may require attention and repair, but, if properly designed, such a structure cannot burst so as to lead to a catastrophe.

Penstock System for Scheme "A"

The arrangement proposed for leading the water from the reservoir to the power house is, in our opinion, not satisfactory. It is expensive and has entailed the introduction of surge tanks. The cost of maintenance of the 90 steel penstocks and of the surge tanks will be high. The penstock system involves the danger of flooding the area between the dam and the power house in case of a burst pipe and necessitates the penstocks to be constructed so as to withstand external pressure and uplift, should a penstock system be empty when the area is flooded.

The elevation of the bottom of the reservoir upstream of the bottom sluices is level with the sill of these sluices. During floods the water through the turbines will therefore carry fine as well as coarse silt, the latter being concentrated to the bottom of the reservoir.

The question of the lowest water level which can obtain in the reservoir without air being drawn into the penstocks has been investigated by Professor E. Meyer-Peter, Zürich, who has carried out extensive model tests in connection with the layout of the penstock system. In Chapter IV of his report of December 20th, 1949, a summary is given of the results of the tests regarding the minimum operating level in the reservoir.

As shown by the tests, the movement of air and floating materials near the intakes is influenced mainly by the height of the water above the inlet openings and by the shape of the intakes.

A hydraulically sound and somewhat streamlined inlet (Intake 2) proved more favourable than a sharp-crested inlet (Intake 1). In the model of Intake 2 scarce, accidental vortices reached into the uppermost pipes when the upstream water level was at R.L. 102.0. When Intake 1 was tested frequent vortices could be observed even at a higher water level, viz. at R.L. 104.0 when the same discharge (183 m^3/sec.) passed the inlet.

In Intake 3 with its projecting piers the flow might be somewhat intensified at the upper edge of the inlet. In this intake floating materials were dragged into the pipes when the water level was at R.L. 100.30; at Intake 2 this happened at water levels lower than R.L. 95.20. Professor Meyer-Peter

recommends that the water level in the reservoir upstream of the dam should never be drawn below R.L. 100.0 as an absolute minimum.

As stated in the beginning of this Report, we suggest R.L. 101.0 as the lowest water level in the reservoir. Should the low water be drawn below this level, the operation of the plant might have to be suspended.

Surge Tanks

The question of turbine regulation and of arranging surge tanks has been the subject of much discussion. Particularly the question whether all units should be equipped with surge tanks has been given special consideration. Regarding this question we have collaborated with Mr. H. Munding of the Swedish State Power Board and Mr. M. Oledal, Professor of Hydraulic Machinery at the Royal Institute of Technology, Stockholm. The results of the investigation may be summarized as follows:-

(a) Stable speed regulation for any one independent unit can only be ascertained if each unit is supplied with a surge tank.

(b) If half the number of main units running at the same time are equipped with surge tanks, and the other units are "blocked", the speed regulation will be satisfactory;

(c) If half the number of main units running at the same time are equipped with surge tanks, and the other units are not blocked, the speed regulation may be satisfactory, even if all units are participating in speed control; and

(d) If none of the main units are equipped with surge tanks and more than half the number of main units running at the same time are blocked, the speed regulation may also be satisfactory.

The turbine manufacturers will only guarantee satisfactory speed regulation for units equipped with surge tanks.

In any case the voltage regulation must not be independent of the speed, if the bulk of the load consists of ohmic resistance or of rectifiers.

Tests carried out for the purpose of this Report at the Wargön Power Station, Sweden, have shown that a satisfactory service is possible with voltage regulation dependent on the speed.

Loss of Head

An estimate has been made of the loss of head from the upstream water level to the entrance of the steel spiral casing located 6.0 m upstream of the centre line of the turbines, where the diameter is 7.8 m, corresponding to an area of 47.76 m^2. Thus, the loss of head in the spiral casings and the draft tubes is not included. The details of the estimate are given in Appendix 2 and may be summarized as follows:-

All units, each for 183 m^3/sec.	1.40 m
KMW units, each for 200 m^3/sec.	1.67 m
EW units, each for 187 m^3/sec.	1.47 m

The discharge of 200 m^3/sec. for a KMW unit and of 187 m^3/sec. for an EW unit corresponds to full gate opening at a water level of R.L. 105.0 in the reservoir and about R.L. 92.5 downstream of the power house corresponding to a net head of about 11.0 m. Since there are 5 KMW units and 2 EW units, the weighted mean of the loss of head is estimated at 1.62 m.

On the basis of model tests, previously referred to, Professor E. Meyer-Peter has estimated the loss of head in the penstocks. In the following table the loss of head is given, as measured in the model, and as estimated by him for the actual penstocks, respectively. The figures have been converted so as to correspond to a discharge of 183 m^3/sec.

The coefficient of the energy loss at the entrance of the flow into and through the bellmouth of the model penstocks is shown in the last column of the table.

Test	Intake	U.S.Level	Loss of Head		Coefficient
		R.L.	measured in model	estimated for actual penstocks	of entrance loss
Drg No.	No.	m	m	m	
2448	1	102.00	1.08	0.80	0.113
2451	1	119.33	1.06	0.79	0.126
2450	1	119.33	1.05	0.79	0.113
2447	1	119.33	1.02	0.78	0.147
2427	1	119.33	1.10	0.85	0.134
2429	1	122.00	1.01	0.78	0.114
2432	2	119.33	1.10	0.86	0.172
2434	2	122.00	1.00	0.78	0.115
2435	3	119.33	1.03	0.77	0.098

It should be noted, however, that turbulence is less developed in the 1 to 18 scale model where Reynold's number becomes 1/76 of that in the full size penstocks. Thus, eddies and discontinuities in flow do not appear to the same extent in the model as will be the case in the actual penstocks. Especially the entrance loss might be somewhat greater than that indicated by the model, where it ranged between 0.10 and 0.17 of the energy head in the 1.8 m diam. pipes. The variation of this coefficient at the different tests indicates that the flow in the model is somewhat unstable and possibly near the transition between laminar and turbulent flow.

In our estimate of the loss of head, Appendix 2, we have assumed a coefficient of the entrance loss of 0.20 which, in our opinion, is the lowest value that can reasonably be assumed.

In Professor Meyer-Peter's Report the friction losses have been calculated on the assumption of a roughness factor of $\lambda = 0.012$ in the actual penstocks. This value corresponds to Manning's number $n = 0.0109$ for the 1.8 m diam. pipes. Swedish practice assumes the value of Manning's number $n = 0.013$. Since VBB's estimate is based on the latter figure, see Appendix 2, the friction losses become somewhat greater than those estimated by Professor Meyer-Peter. Apart from the difference in roughness, owing to the difference in Reynold's number, the friction losses are likely to be greater in full scale than those measured in the model tests.

Estimate of Cost

In Appendix E to their Report of November, 1949, on possible reductions in capital expenditure the Consultants have estimated the saving of capital cost of their Scheme "A", as compared with the Tender Scheme, as follows:-

		£E
Substructure		214,000
Superstructure		178,000
Reduction in thickness of pipe lines		310,000
		702,000
Provisional saving of 3 surge tanks and foundations		291,000
	Total £E	993,000

By introducing such modifications of Scheme "A", as have been suggested by VBB, and which are shown, in principle, on Plate 1, an additional saving of about £E 650,000 can be effected.

A detail estimate of the capital cost of the modified Scheme "A", as now proposed, based on the Consultants' figures and on prices ruling in the autumn of 1948, is given in Appendix 3 and is summarized as follows:-

	£E
Penstock system (mechanical works) according to Tender Scheme	2,839,000
Penstock system (civil works) according to Tender Scheme	698,500
Power house according to Tender Scheme	2,949,700
Sundry works included in Tender Scheme	494,800
Carried forward	6,982,000

	Brought forward	6,982,000
Reductions applicable to modified Scheme "A"		
Modification of:-		
Western approach bridge, cutwater piers, etc.	183,000	
Power house substructure, on basis of VBB's arch and buttress scheme	250,000	
Power house superstructure	400,000	
Rail track system	2,900	
Annex accommodation	20,100	
Reduction of pipe thickness	310,000	1,166,000
		5,816,000
To be added:-		
Sundry works not included in "Scheme "A"		319,000
Customs duty		255,000
Engineering and contingencies 15%		963,000
Works already carried out		912,000
		8,265,000
Further reductions, for omission of		
3 main surge tanks and foundations	291,000	
Contingencies and engineering	44,000	335,000
	Total £E	7,930,000

The annual cost of depreciation and maintenance is given in Appendix 4, and amounts to £E 131,300 per annum. This annual cost, capitalized at an interest of 4 per cent, corresponds to a capital value of about £E 3,250,000.

Conclusions regarding the Tender Scheme
(original or modified)

As drawbacks of the Tender Scheme and Scheme "A" may be mentioned:-

(a) Difficulties of fitting the 90 penstocks into the bottom outlets of the existing dam;

(b) Excavation has to be made into the apron immediately downstream of the dam for all the penstocks;

(c) Air will be drawn into the penstocks at a low water level below R.L. 99.0, in the reservoir, and there will be risk of vibrations being set up in the penstocks owing to contractions at the upper end and/or to retardation of velocity in the conical pipes;

(d) Coarse silt will be discharged with the water through the turbines, thus affecting the cost of maintenance of the turbines;

(e) High maintenance cost of the steel penstocks and surge tanks;

(f) Considerable loss of head in the penstocks;

(g) Owing to structural steel not being manufactured in Egypt, orders for the penstocks and surge tanks will have to be placed abroad;

(h) If three turbines would not be fitted with surge tanks, but be blocked, i.e. the automatic governors would be adjusted so as to prevent any increase in load except after manual adjustment, the closing of the guide vanes at a reduction of load would be slow and the operation of the plant would be somewhat more complicated.

In view of the undesirable features summarized above neither the original Tender Scheme nor the Modified Scheme "A", both elaborated under certain stringent restrictions, can be recommended, and, in our opinion, every effort should be made to find another and a better solution.

4. ALTERNATIVES

The first reaction at a study of the Tender Scheme or the Modified Scheme "A" is, naturally enough, to ask whether the power house could be moved closer to the dam, and whether the bottom outlets in the dam could be enlarged. The water could then be led from the dam to the power house in short concrete conduits at a much lower velocity than in the Tender Scheme, whereby surge tanks could be omitted. With such an arrangement also the spiral casings would be constructed of reinforced concrete in order to get the full benefit of the change.

Careful considerations, however, have indicated that the work of enlarging the bottom outlets would be difficult and might involve certain risks, since the masonry of the original dam in its present state, even if good, may not easily lend itself to this work. For this and other reasons the alternative referred to above has not been proceeded with.

Instead two other possible alternatives are being dealt with in some detail in the following under the headings of

The Forebay Scheme and
The Tunnel Scheme.

In the Forebay Scheme the power house is located at the same place as in the Tender Scheme. The pipe lines and the surge tanks are omitted and, instead, the water is conveyed to the turbines by means of a forebay created by an intake dam close to the power house, and by two wing walls abutting against the dam.

In the Tunnel Scheme all the works for the power production are moved away and located at a safe distance from the dam. The power house is placed about 500 m upstream of the dam. The tunnels for discharging the water, utilized for power production, are located deep into the rock, letting the water out downstream of the dam at about the same place at which the water would be discharged according to the Tender Scheme.

5. FOREBAY SCHEME

General Arrangement

The Forebay Scheme, as outlined at the conference in Cairo on February 6th, 1950, consists of an intake (crest level at R.L. 123.0) and a power house, located downstream of, and parallel to, the existing dam, and of two wing walls connecting the intake to the dam. This arrangement may be referred to as Alternative 1, Plate 2. As a sub-alternative, Alternative 2, to the western wing wall a wall may be constructed parallel to the dam across the western channel, downstream of Set No. 1 in the dam as a connecting wall between the intake and the eastern wall of Lock No. 1, Plate 2.

In Alternative 1 water would be let through 39 sluices in the existing dam into the forebay. In Alternative 2 water to the forebay would in addition be conveyed through eight sluices of Set No. 1 and through one more sluice of Set No. 2. In the latter case new sluices would be arranged in the above-mentioned connecting wall downstream of Set No. 1.

From the forebay water would be conveyed to the turbines through short conduits. Since coarse silt may accumulate at the bottom of the forebay, a silt sluice would be arranged at each turbine. These sluices would be used during flood periods, only, when there is an ample supply of water.

The intake may be constructed either of rubble masonry with granite facing, as shown on Plate 3, or of reinforced concrete, as shown on Plates 4 to 6. In both cases the turbines with their steel spiral casings, as already ordered, would be used.

The two wing walls in Alternative 1 are proposed to be constructed of masonry and reinforced concrete, whilst in Alternative 2 the connection wall between the intake and the eastern wall of Lock No. 1 would be constructed in the same way as the intake, i.e. either of rubble masonry or of reinforced concrete.

The rock at the site of the eastern wing wall is sound. Therefore the excavation need not be deep to reach good rock. In any case this excavation would have to be carried out by hand without using explosives.

The location of the western wing wall in Alternative 1, on the other hand, would require careful consideration, since the rock here is somewhat disturbed. Parallel to the locks there is a major fault line, the approximate position of which is shown on Plate 15, and the wing wall should, if possible, be kept away from this fault. Recent geological examination of the rock in the turbine pits may give a lead as to where the western wing wall should be placed.

In Alternative 2 the eastern lock wall would have to be strengthened, as tentatively shown on Plate 7. Probably no rock excavation for this purpose would be required near the dam, since the width of the foundation of the lock wall appears large enough to allow the wall to be strengthened without any excavation.

Provided the rock excavation for the wing walls be carried out with great care, without using explosives, and that the rock be grouted by new, recently developed methods, the excavation need not be so deep as would have been the case some years ago.

Since the eastern wing wall will occupy the space for House Set No. 11, this unit would have to be moved, e.g. to the position of the spillway for the low level basin in the Tender Scheme. A general study of the most suitable re-location of the units should, however, be made. In this connection it should be borne in mind that it is possible to reduce the distance between the main units from about 30 to about 27 m.

Junction between Wing Walls and Existing Dam

In the Forebay Scheme part of the existing dam would be submerged in water when the power station is in operation. Thus the dam would generally be relieved of water pressure, with the exception of that corresponding to the loss of head, which is small. This part of the dam would then have the same temperature on the upstream as on the downstream side. The existing dam east of the eastern wing wall and west of the western wing wall would, as at present, have water pressure on the upstream side,only, and be subject to variations of temperature due to water on the upstream

side and to air on the downstream side. It is therefore possible that, owing to the difference in stresses and movements at the junction between the wing walls and the existing dam, cracks may develop in the existing dam, if certain precautions were not taken. It is impossible beforehand to estimate the exact position of such possible cracks. For the above reasons it is suggested that permanent expansion joints be arranged in the existing dam at the junctions between the wing walls and the dam. This question requires careful consideration, and a tentative proposal is shown on Plates 7 and 8.

When the power station will be out of operation, e.g. for maintenance of the structures or for other purposes, the sluices in the dam can be closed by means of the existing gates and the forebay completely dewatered.

The joint between the existing dam and the wing wall would be arranged so as to allow of movements of the two structures independently of each other. A cantilever of reinforced concrete from the upstream monolith of the wing wall would abut against the dam, leaving a space of, normally, 60 cm between the upstream face of the cantilever and the dam. At the place of the seal the cantilever would protrude as a tongue entering between two adjacent sliding buttresses of the dam. In order not to interfere with the movements of these buttresses or of the dam itself a space having a width of 2 cm is proposed to be left between the sides of the tongue and the existing structures. It is proposed to obtain this open space by concreting the tongue against rubber mats, placed on the side faces of the buttresses and later on withdrawn. At the downstream face of the dam the tongue would be concreted against layers of bituminous sheeting previously pasted on the dam. It would be sealed by means of stainless steel stops. The watertightness of the seal, would, however, not depend on the steel stops but on bitumen which would be kept under higher pressure than the adjacent water pressure. The bitumen would be enclosed in a slot made through the existing dam, as described below, and would press against a groove in the tongue-tip, thus forming the watertight seal between the wing wall and the dam.

In the existing dam a permanent expansion joint would be arranged by making a row of diamond-drilled holes from above into the sluice at the place of the seal and spaced as closely

to each other as possible. The masonry between the holes would then be removed and the space filled with bitumen which later on would be kept under pressure by means of special pressure vessels, as indicated on Plate 8, and as successfully used at many dams.

The seal would be extended downwards along the side wall of the sluice, at the bottom portion of which a gallery of reinforced concrete would be arranged. From there a row of drain holes would be diamond-drilled into the rock in order to prevent lateral seepage of water from the forebay which otherwise might cause uplift on the existing dam outside of the wing wall. The gate well and the entrance to the sluice would be sealed by means of concrete and the above-mentioned bitumen seal.

The entrances to the two adjacent sluices would also be provided with concrete plugs, but their gate wells would be kept open for drainage purposes. The whole drainage system of the junction would be connected to the main drainage gallery under the wing wall from which the water would be carried to the tail race.

A plan and a cross section of the western wing wall in Alternative 2 are shown on Plate 9.

Surges and Operation of Plant

The forebay would act as a large surge tank and would allow of a very satisfactory regulation of the turbines. In the Forebay Scheme all units would run in the ordinary way with automatic governors, and no units would have to be blocked. If and when the whole station would suddenly stop, for instance owing to a main fault on the transmission system, the water level in the forebay would rise 0.60 m.

Loss of Head

An estimate of the loss of head in the Forebay Scheme is given in Appendix 5. During floods at water level R.L. 105.0 in the reservoir and R.L. 93.7 in the tail race the discharge of a KME unit would be about 200 m^3/sec. and that of an EW unit about 187 m^3/sec. Thus, the total discharge for obtaining firm power would be about 1,375 m^3/sec., excluding the house sets which in this connection may be disregarded. At the discharge mentioned the loss of head would be as follows:-

Alternative 1

Loss through 39 bottom outlets 7.0 m high in the existing dam	0.23 m
Loss in the intake (KMW unit) excluding spiral casing and draft tube	0.13 m
	0.36 m

Alternative 2

Loss through 40 bottom outlets 7.0 m high and 8 outlets 3.5 m high in the existing dam	0.20 m
Loss in the intake (KMW unit) excluding spiral casing and draft tube	0.13 m
	0.33 m

Estimate of Cost

A detail estimate of the capital cost of the Forebay Scheme, based on prices ruling in the autumn of 1948, is given in Appendix 6 and is summarized as follows:-

Masonry Intake Dam, Alternative 1, Steel Spiral Casings

	£E
Intake dam (civil works)	1,768,000
Intake dam (mechanical works)	610,000
East wing wall	270,000
West wing wall	400,000
Power house	2,215,000
Sundry works	387,500
Customs duty	77,500
Engineering and contingencies	860,000
Works already carried out	912,000
Total £E	7,500,000

If the foundation for the east wing wall would be 1 m deeper, and for the west wing wall 6 m deeper than in the above estimate, as suggested by Sir M. Macdonald & Partners, there would be an addition to the above estimate of £E 216,000.

If the intake dam would be carried out as a buttress dam of reinforced concrete instead of as a masonry dam, there would be a deduction of the above estimate of £E 220,000.

If the west wing wall would be carried out according to Alternative 2 (between the intake dam and Lock No. 1), and the foundation taken down to the levels suggested by Sir M. Macdonald & Partners, there would be an addition to the above estimate of £E 110,000.

The annual cost of depreciation and maintenance is given in Appendix 7, and amounts to £E 87,200 per annum for the masonry intake dam, Alternative 1, steel spiral casings. This annual cost, capitalized at an interest of 4 per cent, corresponds to a capital value of about £E 2,200,000.

6. TUNNEL SCHEME

The western bank at the dam appears to lend itself to the construction of a power plant without carrying out any works in close proximity to the dam. In the following a preliminary investigation into the possibilities and economics of such a project, called the Tunnel Scheme, will be made.

General Arrangement

The Tunnel Scheme consists of an intake and a power house located on the western bank about 500 m upstream of the existing dam. Water would be conveyed to the intake by a short head race immediately upstream of the upstream end of the "embankment" for the boat traffic canal leading to the locks. In the power house the same turbines and generators as those on order for the Tender Scheme would be used. A vast depression in the ground would be utilized for the construction of a surge basin immediately downstream of the power house. When no surges are taking place, the variation of the water level in the surge basin would chiefly depend on the water level in the western channel below the dam, where the high water level is estimated at R.L. 97.0 and the low water level at R.L. 87.0, the difference between the water level in the surge basin and that in the channel being the loss of head in the tunnels mentioned below.

Four tunnels would be provided, having their intakes at the northern side of the surge basin. These tunnels would be excavated in solid rock. The average length of the tunnels would be about 820 m. The tunnels would be located so deep in the rock that the water would be discharged underneath Locks Nos. 2, 3, and 4.

The concrete sudd recently built downstream of the dam would also be used in the Tunnel Scheme since the tunnels would, to their whole length, be excavated from the downstream end, and this work would have to go on continuously over a number of years.

The power house would contain 7 main units and 2 house sets, but no provision would be made at this place for low head units. Instead the excavation for draft tubes, already

carried out downstream of the dam, would be used for the installation of low head units in a separate power house.

Since, however, the installation of low head units forms no part of the works to be carried out at this juncture, in this Report the construction of a power house for these units is not considered as part of the Tunnel Scheme proper, nor included in the estimate of cost. Yet, being of considerable interest, the matter is dealt with under a separate heading at the end of this chapter.

The design of the power house of the Tunnel Scheme will depend on whether the spiral casings are to be constructed of steel or of reinforced concrete. If steel spiral casings would be used, the principles of the design for the intake and the power house would be the same as those shown for the Forebay Scheme on Plates 4 to 6. The centre between the units would, however, be reduced to about 27 m, thus allowing a reduction of the length of the power house by about 20 m, as compared with the Tender Scheme. The most economical design, however, would be to use spiral casings of reinforced concrete. The distance between the main units would then be about 21 m, the length of the power house thereby being reduced by about 60 m, as compared with the Tender Scheme. The main transformers would be located downstream of the power house and the switching station west of the surge basin.

A general plan of the plant with steel spiral casings is shown on Plate 10, and the same plant with concrete spiral casings on Plate 11. A longitudinal section is shown on Plate 12.

Since, in our opinion, spiral casings of reinforced concrete would be more economical and render possible a shorter time of construction, steel spiral casings for the Tunnel Scheme are not dealt with any further in this Report, except in regard to the time of construction.

Upstream Temporary Sudds

In order to make it possible to carry out the works on the intake, on the power house, and on the surge basin, it would be necessary to construct a temporary sudd upstream of the intake. Parallel to the river there is a natural ridge on which this sudd may be placed and carried over the entrance of the

depression to the upstream bank. The location of the main sudd is indicated on Plates 10 and 11. The sudd would have a crest level of R.L. 122.0 and may be built as a multiple arch dam with precast reinforced concrete arches. Since the lowest point of the ground under the sudd is at about R.L. 104.0, the sudd can be built entirely in the dry, if executed when there is low water in the reservoir (July to October). The maximum height of the sudd would be 18 m.

To allow most of the excavation of the head race to be carried out in the dry, a temporary auxiliary sudd near the bank of the river would have to be built, having a crest level of R.L. 106.0.

Head Race, Intake, and Power House

The water would be conveyed from the river through a short head race excavated in rock.

The intake would be of the buttress type, built of reinforced concrete. There would be trash racks and hydraulically operated gates. In the power house the water would be led to the turbines through spiral casings of reinforced concrete. The floor of the power house would be located at about R.L. 100.0. At the downstream end of the draft tubes there would be draft tube gates operated by a travelling gantry. The superstructure would be of reinforced concrete.

The main principles of the design of the intake and power house are shown on Plates 13 and 14.

Surge Basin and Tunnels

Water would be conveyed from the power house to the tunnels through the surge basin. Should a fault on the transmission line cause all the units to fall out during a high flood, the water level in the surge basin would first be lowered and would then rise. It is estimated that the maximum rise would be about 1.5 m.

The indications are that the tunnels would be excavated in solid granite. The geology of the ground at the dam site has been investigated by Dr. J. Ball, and an abstract of the geological map is shown on Plate 15. As will be seen from this plate, there is granite at the surface of the proposed site of the surge basin and to the south thereof.

The western flank of the existing dam is excavated deep into the rock, the foundation of the dam being at R.L. 98.0, at which elevation good granite is found. The top of Tunnel No. 1 would be at about R.L. 75.0. Thus, the thickness of the roof of the tunnel at this place would probably be about 23 m.

Locks Nos. 2 to 4 are founded on granite at between R.L. 82.0 and R.L. 87.5. The tunnels would be located at such a depth as would give at least 15 m of cover under the locks, and underneath the locks the roofs of the tunnels would be lined with concrete for a distance of about 50 m, see Plates 12 and 16. The last-mentioned plate shows a longitudinal profile through the locks and the position of the proposed tunnels. Should it be found desirable, for geological reasons, to locate the tunnels at a greater depth than that shown on the above-mentioned plates, there would be no objection to this.

Loss of Head

The loss of head during flood periods, at a discharge of 1,375 m^3/sec., when the upstream water level is at R.L. 105.0, and the downstream water level at R.L. 92.5, is estimated as follows:-

Head race and intake (excluding spiral casings and draft tubes)	0.15 m
Surge basin, tunnels, and tail race	1.24 m
Totalling	1.39 m

An estimate in detail, excluding the losses in the head race, is given in Appendix 8.

Navigation

In order that the power plant may not interfere with navigation, an embankment with crest level at R.L. 123.0 would be constructed on the natural spur of the ground west of the traffic channel, see Plates 10 and 11. Thus the diversion of water from the river into the head race would not cause any difficulty for the traffic to pass into the navigation channel.

Silting

It might be found that silt would deposit in the head race. This question deserves careful consideration. Should such silting take place, it could be removed by dredging. To a certain extent silting might be prevented by arranging special silt sluices at least at both ends of the power house.

In the surge basin and in the tunnels there would be no danger of any serious silt deposits, since, if such deposits should occasionally occur, they would be scoured away when the plant would be operated at its full.

Further Investigations

Before a decision should be taken to carry out the Tunnel Scheme, certain investigation works would be required. The main object of these would be to determine the elevation of the contact zone between granite and sandstone in the area of the proposed tunnels. About 10 diamond-drilled bore-holes would be required for this purpose. The positions of these bore-holes are marked by crosses on Plates 10 and 11. Also a number of exploration test pits and one or two short exploration tunnels should be excavated.

With reference to diamond drilling we have been in contact with Société de Construction des Batignolles in order to find out how quickly the above-mentioned bore-holes could be carried out. A copy of a letter from the Contractor, dated March 8th, 1950, is attached, Appendix 9. Two complete equipments are available and can be shipped to Aswan immediately on receipt of order. It is estimated that the necessary drilling would take about two months after the order has been placed.

A contour survey should also be made of the area to be occupied by the head race, intake, power house, and surge basin.

It is recommended that steps be taken immediately to investigate the Tunnel Scheme by means of diamond drilling, and that a ground survey be carried out forthwith. This should, in our opinion, be done irrespectively of any other decision which may be taken in regard to the hydro-electric project.

Time of Construction

Should the Tunnel Scheme with concrete spiral casings be adopted, the first unit would be put into operation by April, 1954, and the other units would be erected one after another, so that the five first units would be in operation by April, 1955. Should steel spiral casings be used, the corresponding dates would be October, 1954, and October, 1955, respectively. According to the erection programme of January 5th, 1950, the corresponding dates for the Tender Scheme are June, 1953, and April, 1955. It appears, however, that this programme cannot be followed since according thereto the erection of the first draft tube lining should have been started already.

For making it possible to carry out the complete Tunnel Scheme within the same time as would now be possible for the Tender Scheme the following conditions are essential. Order for the excavation of about 700,000 m^3 on the site of the power house and the intake must be placed with the present contractor before May 25th this year. This excavation must be carried out in 8 months and executed from one side of the pit to the other, so that the erection of the draft tube lining for the first unit can be started by December, 1950. The main upstream sudd must be constructed during the low water season of this year and therefore order must likewise be placed with the present contractor. The main contract for the construction works must be let by September this year, and not more than 6 months must be allowed for preliminary site works so that concreting can start by April, 1951.

The tunnelling would be a matter of no difficulty unless very bad rock would be encountered.

Details concerning the construction programme and the methods for executing the excavation and tunnelling will be found in Appendix 10 and on Plates 17 and 18.

Provided that the conditions enumerated above be fulfilled, the actual delay which would be caused by adopting the Tunnel Scheme would be limited to 3 months for putting the first unit into operation; the completion of the whole scheme would be subject to no delay whatsoever, as compared with the Tender Scheme in its present situation.

Any improvements of the programme for the execution of the Tender Scheme, such as improvements concerning the average rate of concreting and the time required for erection of the units and of other mechanical and electrical equipments, may also be applied to the Tunnel Scheme.

Estimate of Cost

An estimate of the capital cost of the Tunnel Scheme, in detail, based on prices ruling in the autumn of 1948, is given in Appendix 12. Since the unit prices for excavation, for masonry, and for concrete, used in the estimate, are the same as those of the present contract, there is little doubt about their accuracy.

In the estimate of cost the unit price for open excavation is taken at £E 1.0 per m^3. Since this excavation to a small extent would consist of sand and to a fairly great extent of sandstone, the remaining part being granite, this figure seems ample.

The inclusive cost in Sweden in 1948 for excavation in hard granite, in tunnels having an area of between 100 and 200 m^2, was about £E 1.3 per m^3. In the estimate of cost of the Tunnel Scheme the cost of tunnel excavation has been taken at £E 2.0 per m^3.

The above unit prices for excavation in open cuts and in tunnels have been confirmed by Société de Construction des Batignolles in their letter of March 7th, 1950, Appendix 11.

The estimate of cost for the Tunnel Scheme is summarized as follows:-

	£E
Temporary sudds	105,000
Head race	500,000
Intake (reinforced concrete)	1,885,000
Power house	1,810,000
Surge basin	530,000
Tunnels	1,140,000
Tail race	210,000
Sundry works	240,000
Customs duty	79,000
Engineering and contingencies	974,000
Works already carried out	912,000
Carried forward £E	8,385,000

		£E
	Brought forward	8,385,000
Reduction for omission of		
7 main steel spiral casings, say		75,000
	Total £E	8,310,000

The annual cost of depreciation and maintenance is given in Appendix 13, and amounts to £E 77,000 per annum. This annual cost, capitalized at an interest of 4 per cent, corresponds to a capital value of about £E 1,925,000.

If steel spiral casings should be used for the Tunnel Scheme instead of spiral casings of reinforced concrete, there would be an additional cost of about £E 500,000.

Low Head Power Plant

As mentioned above, the excavation for draft tubes, already carried out downstream of the dam, would be used for the installation of low head units in a separate power house, on the same principles as those suggested in the Tender Scheme. The number of low head units to be installed in the future should be the subject of a special investigation. It is likely that more than two low head units should be installed, whereby more firm power could be generated in the Tunnel Scheme in combination with a power station at the dam, than would be possible in the Tender Scheme or in the Forebay Scheme. The switching yard constructed so far would be used for the outdoor substation for the low head units.

Since low head units would be used during flood periods only, they could always be operated at constant output, the variation in load being taken by the main units. Kaplan turbines would be unnecessarily expensive, and it is likely that the most suitable kind of turbines would be propeller turbines, since these would have the same efficiency at full load as Kaplan turbines. The speed of a propeller turbine would, however, be considerably higher than that of a Kaplan turbine, whereby a saving in cost of the turbine and the alternator would be effected. Further, it should be taken into consideration whether automatic regulator, etc. could be dispensed with.

In order to facilitate the construction of the power plant for the low head units the eastern portion of the temporary sudd (built below the dam) could be left and continued towards the west by a temporary sudd of rock fill at about the same position as the temporary access road to the tunnels shown on Plates 10 and 11.

7. COMPARISON OF SCHEMES

For comparing the schemes dealt with in this Report not only the capital cost but also the annual cost of depreciation and maintenance should be taken into account as well as the amount of power obtained, the time of construction, and certain other features.

Capital and Annual Cost

It appears practical, for the purpose of comparison, to build up "Comparative Figures" of cost by adding together, for the different schemes, their capital cost and the capitalized value of the annual cost.

The figures referred to are given under the various schemes considered, and are quoted in the table below.

	Capital cost	Capital value of depreciation and maintenance	Comparative Figure
Modified Scheme "A"	7,930,000	3,250,000	11,180,000
Forebay Scheme:-			
Alternative 1	7,500,000	2,200,000	9,700,000
Alternative 2	7,610,000	2,200,000	9,810,000
Tunnel Scheme	8,310,000	1,925,000	10,235,000

As far as the comparative figures are concerned, the Modified Scheme "A" is the least economical. The improvement in economy by adopting the Tunnel Scheme corresponds to about £E 1 million. On the other hand, the difference between the Comparative Figure of the Tunnel Scheme and those of the two alternatives of the Forebay Scheme is about £E 0.5 million, only.

As mentioned above, all estimates of cost are based on prices ruling in the autumn of 1948. No estimates of cost have been made on the basis of the unit prices now ruling.

The cost of the turbines, alternators, and electrical equipment at Aswan has been estimated by the Consultants as follows (excluding customs duty, engineering and contingencies):-

	£E
Turbines and alternators	5,610,000
Cables	230,750
Cranes	276,000
Switch gear, transformers, etc.	2,028,000
Air conditioning	30,000

The cost of the above items would be common for all schemes with two exceptions. Firstly, if spiral casings of reinforced concrete would be used, there would be a rebate on the contract for the turbines due to the omission of the steel spiral casings. This rebate would have to be subject to negotiation and has, for the purpose of this Report, been assumed to be £E 75,000. Secondly, if the main transformers would be placed close to the power house, as proposed in the Forebay Scheme as well as in the Tunnel Scheme, a saving in the abovementioned cost of cables would be made, estimated at about £E 40,000.

Loss of Head and of Power

According to the fluctuations of water levels shown on Contract Drawing C.D. 9, the available gross head during the flood period from about July 20th to about October 10th would vary between 9.2 and 11.0 m during a mean year and would have a weighted mean value of about 9.5 m. This drawing, however, is based on a storage level of R.L. 103.0 during the flood. At the storage level of R.L. 105.0 now adopted for the same period the gross head, decisive for the output of firm power, will be about 11.5 m.

In all the schemes treated the loss of head at 11.0 m net head and a discharge of 1,375 m^3/sec. has been estimated. By converting these figures so as to correspond to a gross head of 11.5 m we arrive at the following:-

	Modified Scheme "A"	Forebay Scheme	Tunnel Scheme
Gross head, m	11.50	11.50	11.50
Loss of head, m	1.47	0.37	1.29
Net head, m	10.03	11.13	10.11
Discharge, m^3/sec.	1,313	1,383	1,324
Total efficiency, per cent	75	76	75
Low tension output, kW	96,800	114,600	98,400

For the purpose of this Report, the output calculated above may be considered as firm power. As will be seen, the gain in firm power by adopting the Forebay Scheme instead of the Tender Scheme would be no less than 17,800 kW. The gain in firm power by adopting the Tunnel Scheme instead of the Tender Scheme would be 1,600 kW. As mentioned above, however, by adopting the Tunnel Scheme there would be a possibility of obtaining considerably more firm power in the future than in any of the other schemes, by installing four, perhaps five, low head units.

On account of air being sucked into the turbines at low water in the reservoir, the loss of firm power in the Modified Scheme "A" may be considerably greater than that given above.

Time of Construction

It is estimated that the Forebay Scheme can be executed in about the same time as the Modified Scheme "A".

With reference to the Tunnel Scheme, which is based upon the use of spiral casings of reinforced concrete, it is estimated that the first unit would be put into operation three months later than the first unit in the Modified Scheme "A" or in the Forebay Scheme, whereas the last unit will be put into operation at the same time as in the other schemes.

General Comparison

A comparison of the three schemes should be made on a broad basis. It should be kept in mind that the total cost of the hydro-electric project will be well above £E 15 millions. It goes without saying that, before such a vast project is embarked upon, there must be no doubt about the suitability and efficiency of the plant.

As mentioned above in this Report, in view of the drawbacks and uncertainties of the Tender Scheme it cannot, in our opinion, be recommended. Besides, the above comparison of costs has shown that it would be less economical than the alternatives considered, due to its high cost of depreciation and maintenance and to its smaller output.

The Forebay Scheme is based on recognized and common practice for utilizing water power. The design put forward for the junction between the wing walls and the existing structure should satisfy all reasonable requirements. It would, however, be necessary to make rock excavation for the wing walls near the existing dam. Provided that this excavation be carried out with great care, without using explosives, and that the rock be grouted by the new, recently developed methods, the excavation need not be so deep as would have been the case some years ago. There should, therefore, be no objection to the Forebay Scheme being accepted.

Also the Tunnel Scheme is based on recognized and common practice for utilizing water power. In this scheme major works near the existing dam would be avoided. According to the estimate of cost this scheme would be somewhat more expensive than the Forebay Scheme.

An attractive feature of the Tunnel Scheme is that it would be possible to install several low head units downstream of the dam, utilizing the present excavation, whereby a greater amount of firm power would be obtained than in any other project. If no change can be made of the present rules of drawing down the reservoir, this feature is of paramount importance, since the economy of the fertilizer plant and the steel works will greatly depend upon the amount of firm power available.

In view of this, too much importance should not be attributed to the difference in capital cost between the Forebay Scheme and the Tunnel Scheme.

It would be a definite condition for the safe operation of the Tender Scheme that the low water level in the reservoir would never be drawn down below R.L. 101.0 (according to Prof. Meyer-Peter R.L. 100.0). On the other hand, both the Forebay Scheme and the Tunnel Scheme could be safely operated at a water level in the reservoir of R.L. 98.0 (or even lower), although of course yielding a correspondingly smaller output.

8. SUNDRY RECOMMENDATIONS

In this chapter certain general recommendations are being dealt with.

Coarse Aggregate for Concrete

During the visits of our representatives to the site coarse aggregate for concrete was crushed and screened in the following grades:-

	Size of particles
Grade No. 1	3.5 to 5.0 cm
Grade No. 2	2.5 to 3.5 cm
Grade No. 3	1.2 to 2.5 cm
Grade No. 4	0.5 to 1.2 cm

Particles less than 0.5 cm were discarded.

For mass concrete it would be suitable to have, in addition to the above-mentioned grades, also a grade with a size of particles of up to, say, 10 cm. The biggest particles now leaving the Pendle Crusher at the site are about 10 cm, and the 5 to 10 cm particles from this crusher should be utilized together with the other grades, for mass concrete.

We understand that it is contemplated to use desert gravel, exclusively, as coarse aggregate for reinforced concrete, but there seems to be no reason for not using crushed granite instead. Therefore, from now on, the different grades should be kept separate from each other.

Fine Aggregate for Concrete

In Chapter 5, page 76, of the Consultants' Specification the limits of grading prescribed for the finest particles of the sand are as follows:-

10 to 30 per cent passing 0.3 mm
0 to 5 " " " 0.15 "

In addition to their importance for obtaining watertightness fine particles are also desirable for obtaining good workability. The result of adding a certain amount of filler, i.e. particles less than 0.075 mm, should also be investigated. In many cases it has been found advantageous to add filler corresponding to 15 per cent of the weight of the cement.

Test of Watertightness of Concrete

For any hydraulic structure subject to unilateral water pressure it is of utmost importance to obtain watertight concrete. In addition to ordinary routines tests of the strength of concrete, its watertightness should also be tested, and the number of routine tests for watertightness should be about the same as those for strength.

Descriptions of two methods of making watertightness tests of concrete are attached, Appendices 12 to 14.

Reinforcement Steel

Some nine years ago the manufacture of indented reinforcement bars of high tensile steel was taken up by a Swedish steel work.[1] It was soon followed by other steel works in Sweden, and similar reinforcement bars are now being manufactured also in the U.S., in France and elsewhere.

Since in any case reinforcement steel must be imported to Egypt, it is recommended that the steel to be used for reinforced concrete should consist of indented bars, the steel having a yield point of 4,000 kg/cm².

Permissible Stresses

In their Report of November, 1949, the Consultants have quoted permissible stresses, stipulated by the Hydro-Electric Power Department, to be applied at Aswan for the purpose of structural design. As far as concrete is concerned, these stresses are about half of what is generally permitted in Sweden for first class concrete. Since it may be taken for granted that the work at Aswan will be carried out by a first class contractor, there is, in our opinion, no reason to prescribe so low permissible

1) E. REINIUS:- "Deformed Bar Reinforcing Steel", The Engineer, December 6th, 1946.

stresses as stipulated. Our experience is that first class concrete can be manufactured in almost any part of the world even with unskilled local labour, provided the work is carried out under first class minute supervision. As far as reinforced steel is concerned, our recommendation is to use indented steel bars having a yield point of 4,000 kg/cm². The permissible tensile stress of such steel would be 1,900 to 2,000 kg/cm².

Resident Engineer

It is recommended that the Consultants appoint a Resident Engineer (Civil) to take up his residence, forthwith, at the dam.

A summary of the contents of this Report is given in the letter of transmission bound with the Report.

Stockholm, March 31st, 1950

For VATTENBYGGNADSBYRÅN (VBB)

A. Frey Samsioe B. Hellström

Nils Berg

ASWAN HYDRO-ELECTRIC PROJECT

Memo
regarding
Floods in the Nile

General Principles

The discharge of water through the Aswan Dam is controlled by gates in the bottom outlets. The controlled run-off equals:-

	(a)	Inflow of surface water to the reservoir
plus	(b)	Inflow of subterranean water to the reservoir
plus	(c)	Rainfall over the reservoir area
minus	(d)	Discharge of subterranean water past the dam
minus	(e)	Evaporation from the reservoir surface
plus or minus	(f)	Storage,

the plus sign before (f) to be used when the reservoir level is falling and the minus sign when the level is rising.

For an approximate calculation of high floods through the Aswan Dam, items (b), (c), (d), and (e) may be disregarded, and (a) "Inflow of surface water to the reservoir" and (f) "Storage", only, have to be taken into account.

Since the amount of storage in the reservoir between low water level R.L. 98.0 and storage level R.L. 121.0 is known, the highest flood that has to be discharged through the dam can be estimated, if the hydrograph (including the peak value) of the inflow to the reservoir during a high flood is known.

Floods of 1817 and of 1818

It appears that reliable records of the Nile floods are available from 1825,[1] [2] showing the maximum height of the annual floods, from that year onwards. In his treatise on the geography of Egypt, however, Dr. JOHN BALL states that extremely high floods took place in the Nile in 1817 and in 1818.[3]

The following extracts from Dr. Ball's treatise may be quoted:-

" ... Belzoni tells us that when he visited the Faiyûm in "May 1819 an extraordinarily high flood had caused such a "quantity of water to pass into the Birket Qarûn that it "rose some 3 1/2 metres higher that it had ever previously "been known to do by the oldest fisherman on its shores; "and the records of the Irrigation Department show that as "lately as 1885 the lake reached to a level more than four "metres higher than it ever attains to-day. The chief "cause of large sudden rises of the Birket Qarûn in the past, "such as the one mentioned by Belzoni, was doubtless the "breaching, during exceptionally high Nile floods, of the "embankments that confine the Bahr Yusef near Hauwaret el "Maqta, whereby a part of the flow in the canal escaped "northwards into the heads of ravines and thence passed "directly into the lake. ... "

From G. BELZONI's narrative of his travels in Egypt, printed in 1822, the following account of a visit to Birket Qarûn on May 1st, 1819, may be quoted:- [4]

" ... Having made an agreement with the owner of the boat, "who might have been named Old Charon himself, we put on "board some provisions, and made towards the west, where "the famous Labyrinth is supposed to have been situated. "The water of the lake was good enough to drink, but a "little saltish: it was only this year that it could be "tasted at all, owing to the extraordinary overflow of the "Nile, which surmounted all the high lands, and, in addition "to the Bahr Yousef, came in such torrents into the lake, "that it raised the water twelve feet higher than ever it "had been remembered by the oldest fisherman among them. ..."

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(1) BENJAMIN BAKER: "The River Nile", Proc. Inst.Civ.Eng., London, 1880, p. 5.

(2) T.W. KEELE: "Investigation of the Nile River Flood-Record for Traces of Periodicity", Proc. Inst.Civ.Eng., London, 1917, p. 5.

(3) JOHN BALL: "Contributions to the Geography of Egypt", Cairo, 1939, p. 231.

(4) G. BELZONI: "Narrative of the Operations and recent Discoveries in Egypt and Nubia", Third Edition, Vol. II, London, 1822, p. 151-152.

In his comments to the above Dr. Ball [3] states as follows:-

" ... The extraordinarily high flood referred to by Belzoni "must have been that of 1818; and ample evidence that the "flood of that year was a phenomenally high and destructive "one is, I find, furnished by the contemporary records of "the Nile gauge at Roda near Cairo, very full extracts from "which were published by ALI PASHA MUBARIK in Part 18 of his "work 'El Khattat el Tewfikia el Gedida' (Cairo 1889) and "translated into French by Prince Omar Toussoun in the "second volume of his admirable 'Mémoire sur l'histoire du "Nil' (Cairo, 1925). These records inform us that in the "year 1233 of the Hegira (1817 A.D.) the Nile rose to such "an extraordinary height at Cairo that the island of Roda "was completely submerged, so that boats could sail over it, "many villages were destroyed, considerable numbers of the "inhabitants and their animals were drowned, and there was "great lamentation among the fellahin over the loss of their "summer crops, especially of the maize crop which formed "their staple food supply; also that in the following year, "1234 of the Hegira (1818 A.D.), there was a still more "disastrous Nile flood, the inundation not only reaching to "even greater heights than in the previous year and sweeping "with much violence over the highest banks and destroying "all the field-crops, including the cotton, as well as the "fruit-trees in the gardens, but also lasting for an abnormally long period, a first slight fall being followed by a renewed "rise to still higher levels after Holy Cross day (Sept. 27), "and the waters not subsiding until the Coptic month of Hatur "(Nov. 10 to Dec. 9), when the season for cultivation was "past. ... "

The statement made by ALI PASHA MUBARIK referred to by Dr. Ball and translated into French by H.R.H. Prince OMAR TOUSSOUN [5] is quoted below:-

" ... En l'an 1233 de l'Hégire (1817 après J.-C.), le "premier dimanche de Shawal (22 Abib), l'ouafà eut lieu; "mais l'ouverture de la digue fut remise à trois jours "après les Fêtes du Ramadan. La cérémonie eut lieu un "jeudi, au milieu d'une foule considérable, et un incendie "se déclara soudainement. Le Katkhoda et le Kadi y étaient. "

" La crue de cette année fut si extraordinaire que le "Nil submergea le coton séfi, le maïs, le sésame, la canne "à sucre, le riz, et la plupart des jardins; ce fut une "véritable mer partout; beaucoup de villages furent détruits, "en même temps que les animaux et les hommes; le fleuve de "Gizeh se mélangea avec celui du Vieux-Caire, les bateaux "naviguaient par-dessus l'île de Rodah. Les fellahs "poussèrent de hauts cris de détresse pour la perte de leurs "cultures, surtout le maïs, leur principal aliment.

(5) S.A. le Prince OMAR TOUSSOUN: "Mémoire sur l'Histoire du Nil", Tome deuxième, Cairo, 1925, p. 510-511.

" Ce jour-là il y eut un incendie, et beaucoup en périrent.
" En l'an 1234 de l'Hégire (1818 après J.-C.), le vendredi
"14 Shaoual (dernier jour d'Abib) eut lieu la proclamation
"publique du cuafà, tandis que le pacha était absent à
"Alexandrie. La crue fut supérieure à l'année précédente,
"et le pays fut submergé deux années de suite; elle se
"soutint jusqu'au mois de Hatour, après l'époque des cul-
"tures; elle baissa ensuite un peu pour remonter plus haut;
"chose étrange, surtout après la fête de la Croix. On avait
"eu soin, pourtant, de réparer les diques, mais rien ne put
"arrêter la violence des eaux. Tout fut noyé comme l'année
"précédente, même le coton, ainsi que les arbres des jardins,
"comme les citronniers et les orangers. L'eau ne s'en alla
"pas promptement, et l'époque des cultures passa; les ouver-
"tures du Khalig furent dépassées et l'inondation se fit une
"voie à travers les terrains du voisinage, comme à Ghet-el-
"Edda et à la Mosquée de l'Émir Hussein et autres. ... "

Estimate of Controlled Flood Discharge

It might be worth while to make an effort to estimate the maximum discharge of the Nile in 1817 and in 1818 on the assumption that the whole island of Roda was submerged. Admittedly, it will meet with difficulties to take various relevant factors into account, for instance the cross section of the Nile as it appeared in these years, but it might, nevertheless, be possible to get an approximate idea of the maximum peak discharge at Cairo. Since certain data of the length of the flood periods are available, approximate hydrographs may also be designed for the floods in 1817 and in 1818. Thereby the factor (a) "Inflow of surface water to the reservoir" would be determined. It would then be a simple matter to estimate the discharges through the Aswan Dam which would then have been required to prevent the water level in the reservoir to rise above R.L. 121.0, or thereabout.

ASWAN HYDRO – ELECTRIC PROJECT

MODIFIED SCHEME "A"

LOSS OF HEAD

Abbreviations :

E.L. Energy Loss
F.L. Friction "
E.S. Entrance Section
I.S. Intermediate Section
F.S. Final "

		Area m^2	Velocity m/s	$\frac{v^2}{2g}$ m	$\frac{v_1^2 - v_2^2}{2g}$ m	Energy Factor	R m	$\frac{R_1 + R_2}{2}$ m	$\frac{v_1 + v_2}{2}$ m	Slope (Friction)	Length m	Friction Loss m	Energy Loss m	Total Loss m	Notes
Trash Racks	E.L.												0.005		Discharge 183 m³/sec
Upstream Face, entrance	E.L.				0.210	0.20 x)							0.042		Manning's Number : n=0.013 (M=77)
E.S.		90.00	2.03	0.210			1.07							0.047	
Bellmouth	F.L.							0.91	2.97	0.0017	3.10	0.005			
"	E.L.				0.565	0.20 x)							0.113		x) Energy factor probably somewhat greater than 0.20 because of turbulent discontinuity in flow at entrance to the penstocks
I.S.		46.80	3.91	0.775			{0.75 0.49							0.165	
Transgression to 1.8 m diam. Pipes	F.L.							0.47	4.95	0.0116	1.60	0.018			
" " " " " "	E.L.				1.045	0.20 x)							0.209		
I.S.		30.54	5.99	1.820			0.45							0.392	
1.8 m diam. Pipes	F.L.							0.45	5.99	0.0176	21.80	0.384			
I.S.		30.54	5.99	1.820			0.45							0.776	
Tapers	F.L.							0.50	4.92	0.0104	14.20	0.148			
"	E.L.				1.067	0.075							0.080		
I.S.		47.71	3.84	0.753			0.56							1.004	
2.25 m diam. Pipes	F.L.							0.56	3.84	0.0054	32.00	0.173			
2.25 " " " (10° Elbows)	E.L.				(0.753)	0.020							0.015		
I.S.		47.71	3.84	0.753			0.56							1.192	
Manifold	F.L.							0.60	3.10	0.0032	2.00	0.006			
"	E.L.				0.473	0.25 xx)							0.118		xx) Energy factor at least 0.25 because of sudden enlargement and thus discontinuities and eddies in flow
I.S.		78.00	2.35	0.280			{0.64 2.18							1.316	
Transgression to 7.8 m square	F.L.							2.07	2.68	0.0005	10.00	0.005			
" " 7.8 " "	E.L.				0.179	0.10							0.018		
I.S.		60.84	3.01	0.459			1.95							1.339	
7.8 m square	F.L.							1.95	3.01	0.0007	7.00	0.005			
I.S.		60.84	3.01	0.459			1.95							1.344	
Transgression to 7.8 m diam. Pipe	F.L.							1.95	3.42	0.0009	7.00	0.006			
" " 7.8 " " "	E.L.				0.286	0.10							0.028		
I.S.		47.76	3.83	0.745			1.95							1.378	
7.8 m diam. Pipe	F.L.							1.95	3.83	0.0011	17.00	0.019			
F.S.		47.76	3.83	0.745			1.95							1.397	
Total loss corresponding to 183 m³/sec discharge:												0.769	0.628	1.397	
" " , KHW units, (200 m³/sec discharge):												0.920	0.750	1.670	
" " , EW " , 187 m³/sec discharge):												0.810	0.660	1.470	

Appendix 3

ASWAN HYDRO-ELECTRIC PROJECT

MODIFIED SCHEME "A"

Estimate of Capital Cost of Civil Engineering and Penstock Work

Estimate based on unit prices ruling in the autumn of 1948 excluding the work associated with the low head units.

		£E	£E
A.	Penstock System (Mechanical) according to Tender Scheme		
	Sluice gates	720,000	
	Penstocks, screens, limpets, etc.	1,390,000	
	Surge tanks (7 main 2 house sets)	562,000	
	Down-stream stop logs (including low head units)	167,000	2,839,000
B.	Penstock System (Civil Works) according to Tender Scheme		
	Penstock area and drains	151,500	
	Surge tank foundations	523,000	
	West wing wall	24,000	698,500
C.	Power House according to Tender Scheme		
	Turbine house foundations	1,135,600	
	Auxiliary construction north of turbine house	544,500	
	Turbine house superstructure	1,078,800	
	Annex superstructure	190,800	2,949,700
	Carried forward		6,487,200

		£E	£E
	Brought forward		6,487,200
D.	Sundry Works included in Tender Scheme		
	Western approach bridge	220,500	
	West bank and filling	130,200	
	Foundations for lock crane	3,300	
	Access roads	40,900	
	Island switching station	88,700	
	Rail trackwork	8,500	
	Water cooling system for cables	2,700	494,800
			6,982,000
E.	Reductions applicable to Modified Scheme "A"		
	Modification of western approach bridge	73,000	
	Modification of cutwater piers and reduction of width of deck	110,000	
	Modification of power house foundations on basis of VBB's arch and buttress scheme	250,000	
	Redesign of power house superstructure on basis of 17.25 m crane span and 16.0 m height from floor to rail using R.C. up to and including crane girder, steel framing above and austerity finishes	400,000	
	Deletion of part of rail track system	2,900	
	Reduction of annex accommodation to a minimum and redesign of framing in R.C.	20,100	
	Reduction of pipe thickness in penstocks	310,000	1,166,000
			5,816,000
F.	Sundry Works not included in Scheme "A"		
	Flapper gates in units for dewatering of penstock area and spiral encasements	25,000	
	Concrete casing to penstocks	186,000	
	Removal of temporary sudd	20,000	
	Excavation to be completed	88,000	319,000
	Carried forward		6,135,000

		£E	£E
	Brought forward		6,135,000
G.	Customs duty		
	Say, 10 per cent on cost of mechanical equipment (£E 2,554,000)		255,000
H.	Engineering and Contingencies 15 per cent		963,000
I.	Works already carried out		
	Temporary sudd	207,000	
	Excavation	370,000	
	Walls and filling and broken stone	335,000	912,000
			8,265,000
K.	Reductions for omission of 3 main surge tanks		
	Surge tanks (mechanical)	225,000	
	Foundations (civil works)	66,000	
	Engineering and contingencies	44,000	335,000
		Total £E	7,930,000

ASWAN HYDRO-ELECTRIC PROJECT

MODIFIED SCHEME "A"

Estimate of Annual Cost of Depreciation and Maintenance

I. Distribution of Capital Cost for Estimate of Annual Cost

Item	Excavation etc.	Concrete masonry etc.	Mechanical equipment	Total
	£E	£E	£E	£E
A.			2,839,000	2,839,000
B.	122,000	576,000		698,500
C.	26,400	2,923,300		2,949,700
D.	75,700	376,400	42,700	494,800
	224,100	3,876,200	2,881,700	6,982,000
E.		856,000	310,000	1,166,000
	224,100	3,020,200	2,571,700	5,816,000
F.	108,000	186,000	25,000	319,000
G.			255,000	255,000
H.	53,000	481,000	429,000	963,000
I.	912,000			912,000
	1,297,100	3,687,200	3,280,700	8,265,000
K.		76,000	259,000	335,000
Total £E	1,297,100	3,611,200	3,021,700	7,930,000

II. Estimate of Annual Cost of Depreciation and Maintenance

		£E
Excavation etc.		
0.35 per cent on £E 1,297,100		4,500
Concrete, masonry, etc.		
1.0 per cent on £E 3,611,200		36,100
Mechanical equipment		
3.0 per cent on £E 3,021,700		90,700
	Total £E	131,300

Abbreviations :

E.L. Energy Loss
F.L. Friction "
E.S. Entrance Section
I.S. Intermediate Section
F.S. Final "

ASWAN HYDRO – ELECTRIC PROJECT

FOREBAY SCHEME, BUTTRES DAM

LOSS OF HEAD

	Area m^2	Velocity m/s	$\frac{v^2}{2g}$ m	$\frac{v_1^2-v_2^2}{2g}$ m	Energy factor	R m	$\frac{R_1+R_2}{2}$ m	$\frac{v_1+v_2}{2}$ m/s	Slope (friction)	Length m	Friction loss m	Energy loss m	Total loss m	Notes
Sluices														
Upstream Face, entrance E.L.				0.127	0.25							0.032		Discharge 1375 m³/sec in 39 sluices 35,26 m³/sec each, Mannings Number n= 0.013 (M=77).
E.S.	22,35	1,58	0.127			1,030							0.032	
Sluice, Part I F.L.							1,070	1,58	0.0004	3.30	0.001			
" " E.L.				0.196	0.20							0.039		
I.S.	14.00	2.52	0.323			0,780							0.072	
Sluice, Part II F.L.							0.780	2.52	0.00150	14.10	0.021			
" "				0.098	0.20							0.020		
I.S.	15.79	2.10	0.225			0.875							0.115	
Sluice, Part III F.L.							0.875	2.10	0.0009	6.40	0.006			
" " E.L.				0.058	0.20							0.012		
I.S.	19.50	1.81	0.167			0.965							0.151	
Sluice, Part IV F.L.							0.965	1.81	0.0006	6.70	0.004			
" F.L.				0.051	0.20							0.010		
F.S.	23.40	1.51	0.116										0.145	
Outlet to Forebay E.L.				0.116	1.00							0.116		
Total loss corresponding to 1375 m³/sec discharge:											0.052	0.229	0.261	
Intake														
Trash Racks E.L.												0.010		Discharge 200 m³/sec, Mannings Number n= 0.013 (M=77).
Upstream Face, entrance E.L.				0.036	0.25							0.009		
E.S.	23.80	0.84	0.036			3.94							0.019	
Bellmouth F.L.							2.90	2.07	0.00018	9.0	0.002			
"				0.516	0.10							0.052		
I.S.	60.84	3.29	0.552			1.95							0.073	
Penstocks, 7.8 m square F.L.							1.95	3.29	0.00075	12.0	0.009			
I.S.	60.84	3.29	0.552			1.95							0.082	
Transgression to 7.8 m dia Pipe F.L.							1.95	3.74	0.00097	6.0	0.006			
" " " " " " E.L.				0.341	0.10							0.034		
I.S.	47.76	4.19	0.893			1.95							0.122	
7.8 m diam. Pipe F.L.							1.96	4.19	0.00121	8.0	0.010			
Total loss, KMW units (200 m³/s discharge):											0.027	0.105	0.132	
" " , EW " (183 " "):											0.024	0.092	0.116	

حضـــرة صاحب السعادة حامد سليمان بك

اتشرف بان ارسل لسعادتكم نسخة من تقرير البروفسور هلستروم عن مشروع توليد الكهرباء من خـــزان اسوان ٠

وتفضلوا بقبول فائق الاحتـــرام ٠٠٠٠

وكيل وزارة الاشغال
لكهرباء القطر المصري

[illegible] / ٥ / [illegible]

<u>Encls</u>:
1 Complete report together with Errata sheets and revised P 41.

8. SUNDRY RECOMMENDATIONS

In this chapter certain general recommendations are being dealt with.

Coarse Aggregate for Concrete

During the visits of our representatives to the site coarse aggregate for concrete was crushed and screened in the following grades:-

	Size of particles
Grade No. 1	3.5 to 5.0 cm
Grade No. 2	2.5 to 3.5 cm
Grade No. 3	1.2 to 2.5 cm
Grade No. 4	0.5 to 1.2 cm

Particles less than 0.5 cm were discarded.

For mass concrete it would be suitable to have, in addition to the above-mentioned grades, also a grade with a size of particles of up to, say, 10 cm. The biggest particles now leaving the Pendle Crusher at the site are about 10 cm, and the 5 to 10 cm particles from this crusher should be utilized together with the other grades, for mass concrete.

We understand that it is contemplated to use desert gravel, exclusively, as coarse aggregate for reinforced concrete, but there seems to be no reason for not using crushed granite instead. Therefore, from now on, the different grades should be kept separate from each other.

Fine Aggregate for Concrete

In Chapter 5, page 76, of the Consultants' Specification the limits of grading prescribed for the finest particles of the sand are as follows:-

10 to 30 per cent passing 0.3 mm
0 to 5 " " " 0.15 "

According to our experience it is difficult to obtain watertight concrete without having a certain amount of very fine particles in the sand. We would therefore recommend that the grading quoted above should be altered as follows:-

15 to 30 per cent passing 0.3 mm
5 to 15 " " " 0.15 "

ASWAN HYDRO-ELECTRIC PROJECT

List of Errata in the Report submitted

Covering Letter

Page 2

Line 36	A special study, etc. ...
read:-	As regards Scheme "A", a special study has been made of the design of the power house with a view to reduce its cost. (To be continued without new line.)

Page 3

Line 1	In Scheme "A" the modifications ...
read:-	In this Scheme the modifications ...

Page 6

Line 6	(in text) ... As mentioned above, however, ...
read:-	However, ...
Line 12	... the loss of firm power in the ...
read:-	... the amount of firm power obtained in the ...
Line 13	... considerably greater than that ...
read:-	... considerably less than ...

Page 7

Line 14	... no objection to the Forebay Scheme ...
read:-	... no technical objection to the Forebay Scheme ...
Line 19	... the Forebay Scheme.
add:-	... the Forebay Scheme. It would also yield less power, if low head units were not installed.
Line 30	... in capital cost between the Fore- ...
read:-	... in capital cost and output between the Fore- ...

Report

Page 4

Line 18 ... over 5,000 cubic metres.

read:- ... over 5,000 million cubic metres.

Page 14

Line 19 ... can only be ascertained if each unit ...

read:- ... can only be obtained if that unit ...

Page 20

Line 9 ... the spiral casings would be constructed ...

read:- ... the spiral casings should be constructed ...

Page 21

Line 8 ... As a sub-alternative, ...

read:- ... In a sub-alternative, ...

Line 9 ... to the western wing wall a wall may be constructed ...

read:- ... the western wing wall is replaced by a wall constructed ...

Page 22

Line 32 ... the dam would generally ...

read:- ... this part of the dam would generally ...

Page 24

Line 34 ... of a KME unit ...

read:- ... of a KMW unit ...

Page 25

Line 20 (heading) Masonry Intake Dam, Alternative 1, Steel Spiral Casings

read:- Alternative 1, with Masonry Intake Dam and Steel Spiral Casings

Page 26

Line 7 ... estimate of £E 220,000.

read:- ... estimate of £E 210,000.

Lines 14 and 15 ... per annum for the masonry intake dam, Alternative 1, steel spiral casings. ...

read:- ... per annum for Alternative 1, with masonry intake dam and steel spiral casings. ...

Page 32

Line 29	... of no difficulty unless ...
read:-	... of no special difficulty unless ...

Page 38

Line 12 (table)	Low tension Output, kW
read:-	Output, kW
Line 24	... the loss of firm power in the ...
read:-	... the amount of firm power obtained in the ...
Line 25	... considerably greater than that ...
read:-	... considerably less than ...

Page 39

Line 23	... be no objection to the Forebay ...
read:-	... be no technical objection to the Forebay ...
Line 29	... Forebay Scheme.
add:-	... Forebay Scheme. It would also yield less power, if low head units were not installed.
Line 33	... firm power would be obtained than in any other project. ...
read:-	... firm power would then be obtained than in any of the other schemes. ...

Page 40

Line 2	... in capital cost between the Fore- ...
read:-	... in capital cost and output between the Fore- ...

Page 42

Line 10	... ordinary routines tests ...
read:-	... ordinary routine tests ...
Line 15	... attached, Appendices 12 to 14.
read:-	... attached, Appendices 14 to 16.

Appendix 10

Page 4

Last line	... the excavation might be carried out ...
read:-	... the excavation beneath the locks might be carried out ...

ASWAN HYDRO-ELECTRIC PROJECT

THE FOREBAY SCHEME

MASONRY INTAKE DAM, ALTERNATIVE 1, STEEL SPIRAL CASINGS

Estimate of Capital Cost of Civil Engineering Work

Estimate based on unit prices ruling in the autumn of 1948.

		£E	£E
A.	Intake Dam (Civil Works)		
	Excavation 45,000 m^3 à £E 1.0	45,000	
	Excavation without explosives 5,000 m^3 à £E 3.2	16,000	
	Rubble masonry and concrete 210,000 m^3 à £E 5.5	1,155,000	
	Shuttering 27,000 m^3 à £E 2.5	67,500	
	Reinforcement, high tensile steel (yield point 4,000 kg/cm^2) 4,000 tons à £E 60.0	240,000	
	Squared rock facing on up-stream and downstream faces 18,000 m^2 à £E 7.25	130,500	
	Moving House Set No. 11	20,000	
	Pressure grouting	30,000	
	Items not specified above	64,000	1,768,000
B.	Intake Dam (Mechanical Works)		
	Gates and hoists for 7 main units, 7 silt conduits and 2 house sets	540,000	
	Trash racks	70,000	610,000
		Carried forward	2,378,000

		£E	£E
	Brought forward		2,378,000
C.	East Wing Wall		
	(Founded at R.L. 85,0)		
	Excavation without explosives 1,000 m³ à £E 3.2	3,200	
	Rubble masonry and concrete 30,000 m³ à £E 5.5	165,000	
	Shuttering 1,000 m² à £E 2.5	2,500	
	Reinforcement, high tensile steel 300 tons à £E 60.0	18,000	
	Squared rock facing 4,000 m² à £E 7.25	29,000	
	Junction to existing dam and expansion joint in the dam	40,000	
	Pressure grouting	4,000	
	Items not specified above	8,300	270,000
D.	West Wing Wall (Alternative I)		
	(Founded at R.L. 80)		
	Excavation without explosives 11,000 m³ à £E 3.2	35,200	
	Rubble masonry and concrete 40,000 m³ à £E 5.5	220,000	
	Shuttering 1,000 m² à £E 2.5	2,500	
	Reinforcement, high tensile steel 300 tons à £E 60.0	18,000	
	Squared rock facing 4,200 m² à 7.25	30,450	
	Junction to existing dam and expansion joint in the dam	60,000	
	Pressure grouting	8,000	
	Items not specified above	25,850	400,000
E.	Power House		
	Turbine house foundations	885,600	
	Auxiliary construction north of turbine house	237,000	
	Turbine house superstructure	678,800	
	Annex superstructure incl. foundations	220,000	
	Carried forward	2,021,400	3,048,000

	£E	£E
Brought forward	2,021,400	3,048,000
Road deck upstream of alternator hall	25,000	
Closing devices for draft tubes	167,000	
Flapper gates and sundry items	36,600	
	2,250,000	
Deduct for modification of anchoring and supporting of spiral casings (see Pl. 3) 7 units à £E 5,000	35,000	2,215,000
Sundry Works		
Western approach bridge	147,500	
West bank and filling	130,200	
Foundations for lock crane	3,300	
Access roads	40,900	
Island switching station	70,000	
Rail trackwork	5,600	
Removal of temporary sudd	20,000	
Excavation to be completed	10,000	
	427,500	
Deduct difference in cost of cables and overhead transmission lines	40,000	387,500
G. Customs duty		
Say, 10 per cent on cost of mechanical equipment (£E 773,600)		77,500
H. Engineering and Contingencies		
15 per cent		860,000
I. Work already carried out		
Temporary sudd	207,000	
Excavation	370,000	
Walls and filling and broken stone	335,000	912,000
	Total £E	7,500,000

Note 1.

	£E	£E
If the foundation for east wing wall is taken down to R.L. 84.0 and for west wing wall to R.L. 74.0 as suggested by Sir M. Macdonald & Partners		
Add for east wall 2,000 m³ à £E 8.7 £E	17,400	
west wall 20,000 m³ à £E 8.7	174,000	191,400
(rate for excavation plus masonry)		
Deduct for pressure grouting		3,400
	net addition	188,000
Engineering and contingencies 15 per cent		28,000
	Total addition £E	216,000

Note 2.

If buttress intake dam is substituted for masonry dam		
Deduct for masonry dam		
Rubble masonry and concrete	1,155,000	
Shuttering	67,500	
Reinforcement	240,000	
Squared rock facing	130,500	1,593,000
Add for buttress dam		
Concrete 130,000 m³ à £E 6.0	780,000	
Shuttering, plain 90,000 m² à £E 2.5	225,000	
Shuttering, curved 10,000 m² à £E 3.0	30,000	
Reinforcement, high tensile steel 6,250 tons à £E 60.0	375,000	1,410,000
	net deduction	183,000
Engineering and contingencies 15 per cent		27,000
	Total deduction £E	210,000

Note 3.

		£E	£E
If west wing wall Alternative 2 (between intake dam and Lock No. 1) is substituted for Alternative 1 and founded as suggested by Sir M. Macdonald & Partners			
Deduct for Alternative 1		400,000	
deep foundation (Note 1)		188,000	588,000
Add for intake dam extension			
Excavation 22,000 m³ à £E 1.0	22,000		
Excavation without explosives 2,000 m³ à £E 3.2	6,400		
Rubble masonry and concrete 62,000 m³ à £E 5.5	341,000		
Shuttering 6,000 m³ à £E 2.5	15,000		
Reinforcement, high tensile steel 500 tons à £E 60.0	30,000		
Squared rock facing 5,200 m² à £E 7.25	37,700		
Gates and hoists for discharge sluices	40,000		
Apron downstream of sluices	30,000		
Pressure grouting	8,000		
Items not specified above	19,900	550,000	
Add for wall alongside lock			
Rubble masonry 19,500 m³ à £E 5.5	107,250		
Squared rock facing 1,800 m² à £E 7.25	13,050		
Pressure grouting	4,000		
Items not specified above	8,700	133,000	683,000
		net addition	95,000
Engineering and contingencies 15 per cent			15,000
		Total addition £E	110,000

ASWAN HYDRO-ELECTRIC PROJECT

THE FOREBAY SCHEME

MASONRY INTAKE DAM, ALTERNATIVE 1, STEEL SPIRAL CASINGS

Estimate of Annual Cost of Depreciation and Maintenance

I. Distribution of Capital Cost for Estimate of Annual Cost

Item	Excavation, etc.	Concrete, masonry, etc.	Mechanical equipment	Total
	£E	£E	£E	£E
A.	61,000	1,707,000		1,768,000
B.			610,000	610,000
C.	3,200	266,800		270,000
D.	35,200	364,800		400,000
E.	26,400	1,985,000	203,600	2,215,000
F.	75,700	351,800	- 40,000	387,500
G.			77,500	77,500
H.	30,000	702,000	128,000	860,000
I.	912,000			912,000
Total £E	1,143,500	5,377,400	979,100	7,500,000

II. Estimate of Annual Cost of Depreciation and Maintenance

	£E
Excavation, etc. 0.35 per cent on £E 1,143,500	4,000
Concrete, masonry, etc, 1.0 per cent on £E 5,377,400	53,800
Mechanical equipment 3.0 per cent on £E 979,100	29,400
Total £E	87,200

ASWAN HYDRO – ELECTRIC PROJECT

TUNNEL SCHEME

LOSS OF HEAD

Abbreviations :

E.L. Energy Loss
F.L. Friction "
E.S. Entrance Section
I.S. Intermediate Section
F.S. Final "

		Area m^2	Velocity m/s	$\frac{v^2}{2g}$ m	$\frac{v_1^2 - v^2}{2g}$ m	Energy factor	R m	$\frac{R_1 + R_2}{2}$ m	$\frac{v_1 + v_2}{2}$ m/s	Slope (friction)	Length m	Friction loss m	Energy loss m	Total loss m	Notes
Intakes															Discharge 700 m^3/sec,
Trash Rocks													0.010		Mannings Number n= 0.013 (M= 77).
Upstream Face, entrance	E.L.				0.012	0.25							0.018		
E.S.		168.76	1.185	0.072			1.65							0.028	
Concrete Penstock	F.L.							1.47	1.83	0.00034	19	0.006			
" "	E.L.				0.239	0.80 x)							0.024		x) Streamlined Flow.
I.S.		81	2.47	0.311			1.28							0.058	
Concrete Penstock	F.L.							1.29	2.47	0.00075	22	0.016			
I.S.		81	2.47	0.311			1.29							0.074	
Acceleration xx)	E.L.				0.582	0.10 x)							0.058		xx) in order to compare with the other schemes with steel casing.
F.S.		47.76	4.19	0.893										0.132	
Total loss, KMW units, (200 m^3/sec discharge):												0.022	0.110	0.132	
" " EW " (187 " "):												0.019	0.097	0.116	
Tunnels															Discharge 1375 m^3/sec,
I.S.		1280 to x) 2330	1.075 to 0.675	0.059 to 0.017			7.3 to 14.7								x) due to changes in downstreams water level from RL 88 to RL 98
Surge Basin	F.L.							10.3 to 17.2	0.806 to 0.495	0.0001	12	0.001			
" "	E.L.				0.044 to 0.006	0.10							0.004 to 0.001		Mannings Number n= 0.040 (M= 25).
I.S.		2560 to x) 4160	0.537 to 0.330	0.015 to 0.006			15.3 to 19.3							0.006 to 0.002	
Upstream Face, entrance	E.L.				0.282 to 0.360	0.25							0.061 to 0.082		4 Tunnels 150 m^2 each.
E.S.		600	2.29	0.267			3.06							0.067	
Tunnel	F.L.							3.06	2.29	0.00106	800	0.848			Manning's Number n= 0.030 (M= 33.3).
" (90° Elbows)	E.L.				0.267	0.20							0.053		
F.S.		600	2.29	0.267			3.06							0.088	
Outlet	E.L.				0.267	1.00							0.267		
Total loss corresponding to 1375 m^3/sec discharge:												0.849	0.386	1.235	

Copy

Société de Construction
des Batignolles
Siège Social
11, Rue d'Argenson,
Paris (VIIIe)

3699

Paris, 8th March 1950

LM/IB

Vattenbyggnadsbyran (VBB)
Humlegardsgatan 29
Stockholm (Suède)

Diamond drilling for Aswan Dam Tunnel Scheme

Dear Sirs,

Following our conversations in your office on 2nd March and according to your request, we are pleased to submit hereunder our offer for two sets of Diamond Drilling Equipment and their operation in view of boring test holes along the proposed lines of the tunnels on the left bank of the Nile at Aswan, Egypt, as shown in your drawings of the "Tunnel Scheme".

The plant we propose will consist of two complete equipments : Craelius AB rotary Diamond drill machines for diameters 55 to 45 m/m each, capable of drilling to a depth of 80 M. and utilising Diamond drill bits for boring through siliceous rock (Nubian sandstone and granite).

These equipments will comprise each : a Craelius AB rotary drill machine, a feeder pump, an electric motor, the sets of drill rods and core extractors and all necessary spare parts, fittings and tools.

All this plant is available immediately in France and may be shipped to Aswan as soon as we receive the order.

The specialised personnel we consider sending to the site will comprise one chief Foreman and four operators (one per shift for each machine); the work will be executed at the rate of two eight hour shifts daily.

Our offer consists of :

1°) A lump sum covering :

- Cost of preparation and transportation from Paris to Aswan and return -
- Hire cost of equipment during transportation -
- Custom and excise duties, insurance, etc. -
- Travel expenses for personnel from Paris to Aswan and return -

Lump sum per machine .... £.E. 900.

2°) A unit price per meter run of drilling in granite to a maximum depth of 80 m.; this price includes the hire cost of equipment when working at site, the salaries of European personnel and local labour and the cost of diamond drill bits, diameter 46 m/m.
The rate of drilling in granite is approximately 3 m. per 8 hour shift -

Per meter run in granite .. £.E. 10.6

3°) A unit price per meter run of drilling in Nubian sandstone; this price includes the hire cost of equipment when working at site, the salaries of European personnel and local labour and the cost of diamond drill bits diameter 46 m/m.
The rate of drilling in Nubian sandstone is approximately 4 m. per 8 hour shift -

Price per meter run in Nubian sandstone .. £.E. 9.4

4°) If eventually it is necessary that a Drilling Engineer should visit the works, the travel expenses and salaries of such an Engineer would be added to and are not provided for in the above.

Hoping we may be of service to you, we remain

Yours faithfully,

Sd/ Ernest Goüin
Le Président
Directeur Général

ASWAN HYDRO-ELECTRIC SCHEME

COMMENTS ON THE CONSTRUCTION PROGRAMME

(Plates 17 and 18)

Should the Tunnel Scheme be adopted, the work on the upstream sudd should start in June, 1950. As previously mentioned, the lowest point of the ground at the sudd upstream of the power station is at about R.L. 104. Generally, the water level in the reservoir has fallen to this elevation by the middle of July. Everything should be ready for the construction of the lowest portion of the sudd at the beginning of July, 1950. This lower portion must be completed by the beginning of October, 1950, when the water level may rise above R.L. 104.

It is suggested that, in order to expedite matters, the sudd as well as the excavation for part of the head race, for the intake, for the power house, and for part of the surge basin be entrusted to the same Contractor and carried out under the same contract as the work so far carried out for the Tender Scheme. Société de Construction des Batignolles, the present Contractor, have estimated this excavation, comprising a quantity of about 700,000 m^3, to be completed within 8 months from the date when they receive the order, provided this is given to them before May 25th, 1950. Thus, if the order is placed before this date, the excavation can be completed by February, 1951.

It would then be possible to start the erection of the steel lining of the first draft tube by December, 1950, the excavation being completed gradually from one side of the power house to the other. Five months would be required for this erection which would be completed by May, 1951. Then the concreting of the substructure could be started. The further construction work for and the erection of the first unit to be put in operation would be as follows:-

The construction of the substructure, including the concrete spiral casing, would require 12 months, which, because of the spiral casing being of concrete, is 4 months more than the time calculated for the Tender Scheme. The substructure, Stage I, would thus be completed by May, 1952. The superstructure and the substructure, Stage II, should be constructed simultaneously and could be completed in 5 months, i.e. by October, 1952. The time required for the superstructure would be 2 months less than the time calculated for the Tender Scheme, due to the smaller quantity of concrete.

The erection of the turbine would have to start with the stays which should be erected before the concrete spiral casing is constructed, so that the stays will serve as supports for the shuttering of the spiral and for the spiral structure itself.

The further erection of the turbine and alternator would require a time of 18 months after the completion of the substructure, which is 4 months shorter than the time required in the case of steel spiral casings. Thus, the first unit could deliver power by April, 1954. The other units would be erected one after the other, so that the five first units would be running by April, 1955.

It is presumed, however, that a travelling crane can be used from the very beginning of these 18 months. Consequently, one of the travelling cranes would have to be erected at an early date, so that it would be ready to operate over the first pit by October, 1952. The cranes could be erected over the assembly bay where the superstructure should have been constructed at an early stage.

In case steel spiral casings would be used for the Tunnel Scheme, the time required for completing the construction and erection work for the first unit would be the same, since the construction of the spiral casing is calculated to take the same time either it is made of steel or of reinforced concrete.

The concreting of the intakes could be started in April, 1951, provided that 6 months, at the most be allowed for preliminary site works and provided that the contract be

placed in September, so that the preliminary works could be started in October, 1950.

The main upstream sudd would have to be removed before October, 1952, since certain under-water excavation for the head race would have to be made, at the latest, during the next low water season (in 1953).

By October, 1952, when the main upstream sudd would have been removed, the upper water level will rise above R.L. 104.0. Therefore, all the intakes would have to be completed by July, so that all the gates might be erected before October. If necessary, the intake openings could be temporarily shut off by stop logs on the trash racks while the gates behind them are being erected.

Thus the total time for construction of the intakes would be restricted to about 16 months, and the quantity of concrete to be placed each month would be some 7,500 m^3, which is about the same rate of work as that calculated for the Tender Scheme.

If steel spiral casings would be used, the total quantity of concrete in the intakes would be greater. At the same rate of work the time for construction of the intakes would then be 20 months, and it would not be possible to remove the main upstream sudd before the low water season of 1953. In such a case the first unit could not be put into operation before October, 1954. Thus the use of steel spiral casings would delay the first power delivery by 6 months, unless means were provided for increasing the rate of concreting from 7,500 to 9,500 m^3 per month. Even then it would not be possible, however, to reduce the total time of construction by more than 3 months, because of the considerably greater amount of excavation to be carried out before concreting could be started.

For the excavation of the remaining portion of the head race an auxiliary sudd with the crest on R.L. 106.0 would have to be built during the period July to September, 1951. In June, 1952, the area behind this sudd would be dewatered. The outermost part of the head race would have to be excavated under water, which should be done in the low water seasons of the following years, by blasting and removing the rock by means of draglines.

The remaining portion of the surge basin, which would be left when excavating for the power house, should be excavated as soon as the excavation for the power house would be completed. A minor part would have to be left as an access road for the trucks until one of the tunnels could be used for the transport of rock.

The excavation of the four tunnels would be started at the downstream end where it is easy to reach the tunnel exits by a road. Later on the excavation for the road would become part of the tail race.

The excavation of a tunnel of the size contemplated can be carried out either according to the full-face method or in two stages.

In the case of full-face excavation the drilling is made from a drilling jumbo, mounted on a carrige which, after the completion of a series of bore-holes, is removed from the tunnel face and placed at a certain distance during blasting and loading. The work is generally carried out in three shifts, boring and blasting during one shift and loading during the following two shifts. The progress of such a tunnel excavation is about 4 m per day.

In the case of the excavation being carried out in two stages, the lower half of the tunnel may be excavated to its whole length in the first stage and the upper part of the tunnel in the second stage. For the excavation of the lower half the full-face method is used as described above but applied to the smaller section, only. In the second stage, while the top portion of the tunnel is excavated the rock blasted during one shift will occupy the lower two-thirds of the area of the tunnel. The boring takes place from the heap of rock thus remaining from the previous blasting. Loading and transport are carried out independently of boring and blasting and may be continued during three shifts. The rate of progress in the second stage is about 8 m per day.

When excavating the portions of the tunnels beneath the lock, as well as the portions less than 80 m from the foundation of the dam, special care would have to be taken to reduce vibrations when blasting, and to reduce the risk of rock falling out to a larger section than intended. Therefore, the excavation might be carried out as follows. At

first small galleries located at the upper corners of the cross-section of the tunnel should be excavated. At intervals interconnections between these galleries should be excavated, the remaining rock between the interconnections temporarily supporting the ceiling. Concrete arches should then be made in the interconnections to support the ceiling. Afterwards the rock remaining between the galleries should be excavated. When this excavation has been carried out additional concrete arches should be made between the former ones, so that all the tunnel ceiling would be supported by concrete arches. Finally, when the upper portion of the tunnel is excavated and its ceiling is lined with concrete in this manner, the lower portion would be excavated.

Either the excavation is carried out with the full-face method, or in two stages, there would be no lack of time for the excavation and lining of the tunnels. On the contrary, there would be plenty of time unless very bad rock were encountered, the date for putting the first unit into operation being determined by other works. Therefore ample time is available for further consideration of the exact location of the tunnels which should not be finally determined until a thorough survey has been finished.

The main feature of the construction programme for the Tunnel Scheme, as shown on Plate 17, is that the excavation work for the power house would be started in July, 1950, by the present Civil Engineering Contractor, who would also have to be entrusted with the construction of the main temporary sudd across the head race. The erection of the steel sheet lining for the draft tubes should start in December, 1950. The concreting of the intakes should start in February, 1951, and be carried out be the Main Contractor who need not necessarily be the same as the present Contractor.

The first five units will be put into operation successively from April, 1954, to April, 1955, if concrete spiral casings are used, and from October, 1954, to October, 1955, in the case of steel spiral casings, see Plate 18.

The programme of January 5th, 1950, for the erection of turbines and alternators in the Tender Scheme shows that the first five units would be put into operation successively

from June, 1953, to April, 1955. Thus the date for the first power delivery would be 10 months earlier than in the Tunnel Scheme, the date for completion of the first five units being the same in both schemes. It appears, however, that this erection programme could be followed only on condition that one of the travelling cranes could be in operation over the pit for the first unit by September, 1951. This would be possible only if the concreting of the substructure could begin by June, 1950. In that case the erection of the draft tube lining for the first unit should have been started by January this year. There seems to be no possibility whatsoever of starting concreting before December, 1950, and, consequently, the erection programme for the Tender Scheme would be delayed by at least 6 months on account of the present stage of the works.

The actual delay which would be caused by adopting the Tunnel Scheme would thus be limited to 3 months. This delay, however, refers to the first unit to be put into operation, only; the completion of the whole scheme would be subject to no delay whatsoever, as compared with the Tender Scheme in its present situation.

Any improvements of the programme for the execution of the Tender Scheme, such as improvements concerning the average rate of concreting and the time required for erection of the units and of other mechanical and electrical equipments, may also be applied to the Tunnel Scheme.

Copy

Société de Construction
des Batignolles
Siège Social
11, Rue d'Argenson,
Paris (VIIIe)

3488

Paris, le March 7th, 1950

LM/IB

Vattenbyggnadsbyran (VBB)
Humlegardsgatan 29
Stockholm (Suède)

Aswan Dam Power Station Scheme

Dear Sirs,

You have been kind enough to communicate to us your ideas on the Aswan Dam Power Station Scheme and to ask for our opinion as to the prices and time according to which this work could be executed.

1) Concerning the excavations for the Power House, which amount to 690.000 m³ if the scroll cases are in reinforced concrete, and to 820.000 m³ if the scroll cases are made of steel, we agree in either case to execute the work according to the unit prices of our present contract "Excavation and Sudd for Aswan Dam Hydro-Electric Scheme", increased by 20 % which represent the increase in price levels between August 1948 and March 1950. The unit price of the most important item : No 20, thus becomes :

L.E. 1,034 x 1,20 = L.E. 1.24 per m³.
(One L.E. twenty four per m³)

The time of execution for this portion of the excavation would be eight months in the first alternative (reinforced concrete cases) and ten to eleven months in the second alternative (steel scroll cases), starting from the date when we receive the order to commence the works. However, in order to enable us to observe this time limit, a decision should be taken before the 25th May, 1950, as a commitment is pending for our equipment at this date.

2) We could execute the tunnels, having an area of 135 m², at a unit price of :

L.E. 2,35 per m³ (Two L.E. thirty five per m³.)

Under the assumption that the rock has a solid consistency, avoiding any timbering and excessive dewatering, which is the supposition indicated to us.

Concerning the 40.000 m³ required for passing under the locks, we would be obliged to increase the unit price by L.E. .25 to .50 per m³, as special means of execution must be taken for this portion of the job.

The time of the execution of the four tunnels would be : two years.

3) We would execute all the additional open mass excavations provided for in your programme at the rate of :

L.E. 1,24 per m³ (One L.E. twenty four per m³)

which is the same as that of item 20 of our present contract modified in § 1) of this letter.

The total time of execution for this portion of the job is approximately : two years.

All prices in this offer are calculated according to present day price levels, and with the assumption that at least half of the specialized labour such as: drillers, mechanics and engine drivers, may be allowed to come from the Contractor's European personnel.

We wish to add in this letter and for your information that we have examined the alternative with reinforced concrete scroll cases you submitted to us and we are satisfied that no special difficulty will be encountered for the execution of such an alternative.

Hoping that we may be of service to you, we remain

Yours faithfully,

Sd/ Ernest Goüin
Le Président
Directeur Général

ASWAN HYDRO-ELECTRIC PROJECT

TUNNEL SCHEME

Estimate of Capital Cost of Civil Engineering Work

Estimate based on unit prices ruling in the autumn of 1948.

			£E	£E
A.	Temporary Sudds			
	Upstream sudds, including removal		85,000	
	Dewatering		20,000	105,000
B.	Head Race			
	Excavation:-			
	in the dry	380,000 m³ à £E 1.0	380,000	
	under water	40,000 m³ à £E 3.0	120,000	500,000
C.	Intakes			
	Excavation	170,000 m³ à £E 1.0	170,000	
	Concrete	90,000 m³ à £E 6.0	540,000	
	Shuttering	100,000 m² à £E 2.5	250,000	
	Reinforcement steel, indented bars, (yield point 4,000 kg/cm²)	4,500 tons à £E 60.0	270,000	
	Items not specified above		30,000	
	Pressure grouting		15,000	
	Gates and hoists		540,000	
	Trash racks		70,000	1,885,000
D.	Power House			
	Excavation	270,000 m³ à £E 1.0	270,000	
	Turbine house foundations:- (concrete spiral casings)			
	Concrete	72,500 m³ à £E 6.0	435,000	
	Shuttering	70,000 m² à £E 3.0	210,000	
	Reinforcement steel	5,000 tons à £E 60.0	300,000	
			945,000	
	Carried forward		1,215,000	2,490,000

		£E	£E
	Brought forward	1,215,000	2,490,000
Turbine house superstructure:-			
Concrete	8,000 m³ à £E 6.0	48,000	
Shuttering	20,000 m² à £E 2.5	45,000	
Reinforcement steel	500 tons à £E 60.0	30,000	
Wall facing, etc.	5,000 m² à £E 4.0	20,000	
Roof trusses		35,000	
Crane girders		75,000	
		253,000	
Items not specified above on sub- and superstructures		38,000	
Annex buildings		170,000	
Closing devices for draft tubes		134,000	
		342,000	1,810,000
E. Surge Basin			
Excavation	530,000 m³ à £E 1.0	530,000	530,000
F. Tunnels			
Tunnel inlets, including semi-circular stop logs		30,000	
Excavation:-			
ordinary	380,000 m³ à £E 2.00	760,000	
below locks	30,000 m³ à £E 2.35	70,000	
near dam	30,000 m³ à £E 2.35	70,000	
Concrete lining, say,		180,000	
Tunnel outlets, including semi-circular stop logs		30,000	1,140,000
G. Tail Race			
Excavation:-			
in the dry	150,000 m³ à 1.0	150,000	
under water	20,000 m³ à 3.0	60,000	210,000
H. Sundry Works			
Access roads, unloading wharf, etc.		130,000	
Outdoor switching station, etc.		90,000	
Removal of downstream sudd		20,000	240,000
	Carried forward		6,420,000

		£E	£E
	Brought forward		6,420,000
I.	Customs Duty		
	Say, 10 per cent on cost of mechanical equipment (£E 794,000)	79,000	79,000
K.	Engineering and Contingencies		
	15 per cent on £E 6,499,000		974,000
L.	Works already carried out		
	Temporary sudd	207,000	
	Excavation	370,000	
	Walls, filling and broken stone	335,000	912,000
			8,385,000
M.	Reduction		
	for omission of 7 main steel spiral casings, say,	75,000	75,000
	Total	£E	8,310,000

ASWAN HYDRO-ELECTRIC PROJECT

TUNNEL SCHEME

Estimate of Annual Cost of Depreciation and Maintenance

I. Distribution of Capital Cost for Estimate of Annual Cost

Item	Excavation etc. £E	Concrete masonry etc. £E	Mechanical equipment £E	Total £E
A.	105,000			105,000
B.	500,000			500,000
C.	170,000	1,105,000	610,000	1,885,000
D.	270,000	1,406,000	134,000	1,810,000
E.	530,000			530,000
F.	900,000	190,000	50,000	1,140,000
G.	210,000			210,000
H.	120,000	120,000		240,000
I.			79,000	79,000
K.	420,000	423,000	131,000	974,000
L.	912,000			912,000
Total £E	4,137,000	3,244,000	1,004,000	8,385,000

II. Estimate of Annual Cost of Depreciation and Maintenance

	£E
Excavation, etc.	
0.35 per cent on £E 4,137,000	14,500
Concrete, masonry, etc.	
1.0 per cent on £E 3,244,000	32,400
Mechanical equipment	
3.0 per cent on £E 1,004,000	30,100
Total	£E 77,000

ASWAN HYDRO-ELECTRIC PROJECT

Methods of making Watertightness Tests of Concrete

Watertightness tests may be made in accordance with either of the two methods described below.

Method 1

Test Specimen and Mould

The test specimen shall have the shape of a hollow octagonal prism with a height of 50 cm and a distance between two opposite outer sides of 30 cm. The hollow shall be a cylinder with a diameter of 10 cm.

The test specimen shall be formed in a wooden mould, octagonal in form (see Appendix 15). The mould shall be substantial enough to maintain its form during the moulding of the test specimen. It should not vary from the standard shell thickness and internal diameter by more than 3 mm nor from the standard length by more than 15 mm. The mould shall be kept saturated with water for no less than 24 hours and for no longer than 48 hours and it shall be dried before the specimen is formed. The mould shall have a core 10 cm in diameter.

Samples of concrete for test specimens shall be representative of the concrete used in construction. Stones larger than 5 cm shall be removed.

If the concrete is not ready-mixed for general use, it shall be mixed either by hand or in a suitable laboratory mixer, in batches of such size as to leave about 10 per cent excess after moulding of the test specimens.

In the case of hand-mixing the batch shall be mixed in a watertight, clean, damp, metal pan, with either a blunt brick-layer's trowel or a shovel, whichever is more convenient. The procedure shall be as follows:-

The cement and the fine aggregate shall be mixed until they are thoroughly blended. The dry mixture shall be spread to a uniform layer, the coarse aggregate shall then be uniformly

spread over it, and the entire batch shall be mixed. Water shall be added and the mass be mixed until the concrete is homogeneous in appearance and has obtained the desired consistency. If prolonged mixing would be required, because of successive addition of water in small quantities while the slump is adjusted, the batch shall be discarded and a new batch made.

In general, for laboratory tests hand-mixing shall be used unless a really suitable laboratory mixer is available.

If machine-mixed concrete shows signs of segregation, it shall be deposited in a watertight, clean, damp, metal pan and re-mixed with a trowel or a shovel.

Simultaneously with the mixing of the concrete to be tested, mortar shall be prepared. The proportions of dry materials in the mortar shall be 1 part of cement to 2 parts of sand suitable for use in high-strength concrete. The water-cement ratio of the mortar shall be approximately 0.5. The mortar shall be used for making watertight layers at both ends of the test specimen.

The test cylinder shall be formed with its axis vertical. The core shall be placed centrically in the external mould and held precisely in the centre until the mould is filled with concrete. A sealing layer of mortar, approximately 8 cm thick, shall be placed first, followed by a 8 cm thick layer of concrete. Both layers shall be tapped simultaneously, in order to avoid cracks between them. The concrete shall thereafter be placed in the mould in equal layers, approximately 8 cm thick. In placing each scoopful of concrete, the scoop shall be moved around the top edge of the mould so that the concrete will slide from it along the mould, in order to ensure an even distribution of the concrete. Each layer shall be tapped with 40 strokes of a 5/8 in. straight round steel rod, approximately 1 m in length and with a spherically shaped base surface. The strokes shall be distributed uniformly over the cross section of the mould and shall be just so heavy that the rod penetrates slightly into the underlying layer. The top layer, approximately 8 cm thick, shall be made of the watertight mortar and rammed so as to form a compact mass together with the underlying concrete.

When the concrete has become just hard enough, the core shall be removed and the internal surface roughened thoroughly with a steel brush. If wooden cores have to be used as substitutes for the composite steel cores, shown on the drawing, Appendix 15, they shall be slightly conical (diameters 9 cm to 10 cm) and saturated with water. If such cores, or simple steel tubes, are used, they shall be removed about 6 to 8 hours after moulding. If composite steel cores are used, they may be kept in their places for 24 hours.

Storing and Curing

During the first 4 days the test specimens shall be stored in a moist condition at a temperature of approximately 20°C. Immediately after moulding they shall be covered with a double layer of wet burlap which shall be kept wet until they are removed from the moulds after 4 days. Specimens must not be exposed to running water. Attention is drawn to the fact that, since evaporation takes place, the temperature within damp sand and under wet burlap or similar materials will always be lower than the temperature in the surrounding atmosphere. At the end of the curing period (4 days) the specimens shall be stored in open air.

If the specimens cannot be adequately tested at the site they shall, at the end of the curing period, be sent to an institutional laboratory.

After curing, and in time before testing, both ends of the hollow of the specimen shall be plugged with watertight mortar such as specified above. For this purpose the test specimens shall be placed on moist paper. In the bottom of the hollow 2 layers of the watertight mortar shall be placed to give the plug a thickness of 10 cm. This mortar shall be lightly rammed with a wooden stick until water becomes visible on the surface of the mortar, but no longer. Water must not separate from the mortar. Watertight mortar for the top plug shall be placed in 2 layers on a round plate fixed in its place 10 cm from the top of the specimen and provided with a pipe for the purpose later on of supplying the hollow with water under pressure. The mortar shall be lightly rammed in the same manner as the mortar in the bottom plug.

Testing

Internal water pressure shall be applied by connecting the pipe leading to the hollow of the test specimen to a water supply pipe. The water shall be clean, free from silt and, if necessary, filtered. The water pressure shall be 5 kg/cm², and it shall be applied for the first time, when the test specimen is 7 days of age. The pressure shall be maintained for 2 days. During the test the specimen shall be kept in a horizontal position.

If the test specimen is entirely dry on the outside at the end of this first test, it shall be accepted as watertight and not further tested. In case wet spots or leakage should be observed, the test specimen shall be disconnected from the water supply and stored in open air. When the test specimen is 28 days of age it shall be tested, for the second time, at the same water pressure for another 2 days. Such specimens as at the end of this second test are entirely dry on the outside shall be accepted as watertight, specimens then showing wet spots or leakage shall be classified as not watertight

Method 2

Test Specimen and Mould

The test specimen shall have the shape of a cylinder with a height of 30 cm and a diameter of 21 cm. A porous core (e.g. of highly porous brick) with a height of 7.0 cm and a diameter of 7.0 cm, and joined to a pipe (for supplying water under pressure) shall be centrically embodied in the specimen (see Appendix 16). The mould may conveniently be made of 1 mm tinned steel plate fixed on a wooden base.

The test specimen shall be made by placing the concrete in three layers, each layer being approximately one third of the height of the mould. Each layer shall be tapped with 25 strokes of a 3/4 in. steel bar 50 cm long. The strokes shall be distributed in a uniform manner over the cross section and shall be just so heavy that the rod slightly penetrates into the underlying layer. After the second layer has been tapped, the porous core, thoroughly saturated with water and provided with the pressure pipe, shall be carefully placed in the centre of the mould. The top layer shall then be placed and tapped and the surface struck off with the bar.

Storing and Curing

The test specimen shall be stored and cured in the same manner as stated under Method 1. The moulds shall be removed just before testing.

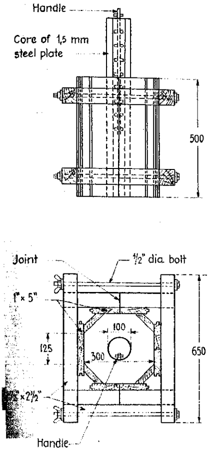
Testing

The testing of the specimen by applying internal water pressure shall be carried out in the same manner as stated under Method 1 and the results thereof interpreted accordingly.

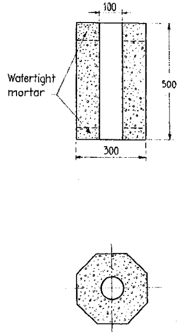
ASWAN HYDRO-ELECTRIC PROJECT

MOULD FOR WATERTIGHTNESS TESTS

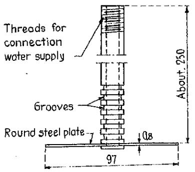
METHOD 1



Wooden mould having a cylindrical steel core



Watertightness test specimen
Scale 1:12

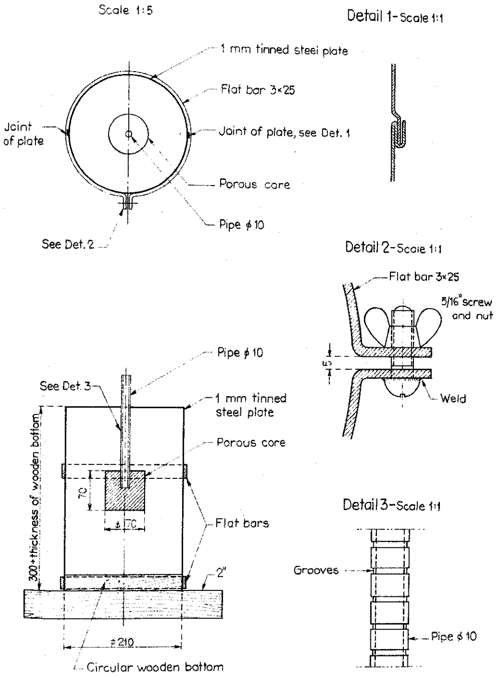


Pressure pipe. Scale 1:2

ASWAN HYDRO-ELECTRIC PROJECT

MOULD FOR WATERTIGHTNESS TESTS

METHOD 2



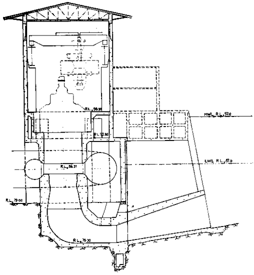
ASWAN HYDRO-ELECTRIC PROJECT

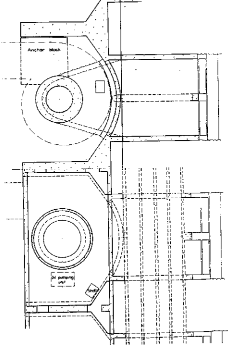
POWER HOUSE

ORIGINAL PRINCIPLES OF VBB DESIGN FOR STEEL SPIRAL CASINGS

Scale 1: 200

Metres 5 0 5 10 15 20 Metres





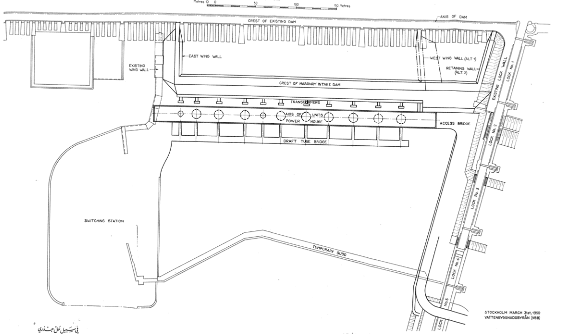
STOCKHOLM DECEMBER 13th, 1949
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

THE FOREBAY SCHEME

GENERAL PLAN

Scale 1:1000



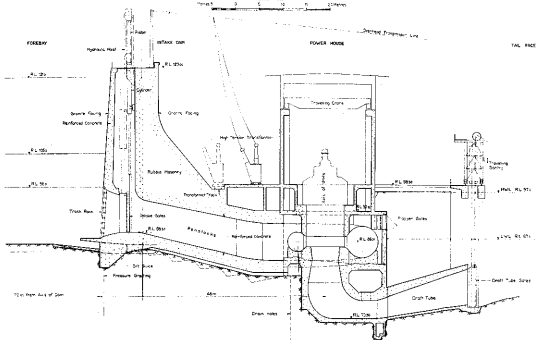
STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

THE FOREBAY SCHEME

MASONRY INTAKE DAM

CROSS SECTION THROUGH POWER PLANT



STOCKHOLM MARCH 31st 1950
VATTENBYGGNADSBYRÅN (VBB)

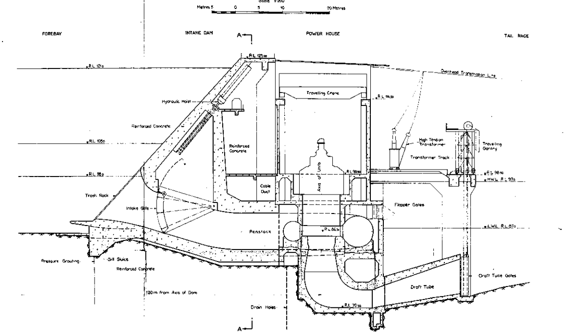
ASWAN HYDRO-ELECTRIC PROJECT

THE FOREBAY SCHEME

BUTTRESS INTAKE DAM

CROSS SECTION THROUGH POWER PLANT

Scale 1:200



STOCKHOLM MARCH 31st 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

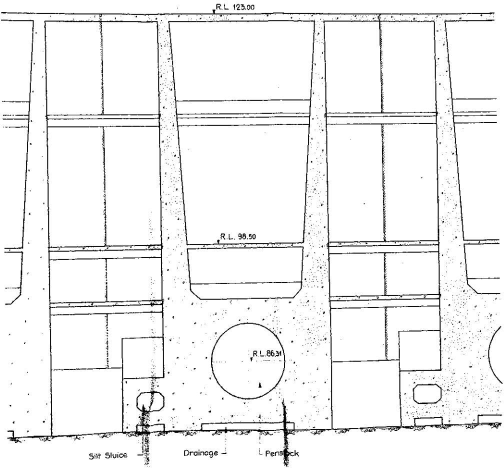
THE FOREBAY SCHEME

VERTICAL SECTION THROUGH BUTTRESS INTAKE DAM

Scale 1 : 200

Metres 5 0 5 10 15 20 Metres

SECTION A-A



STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

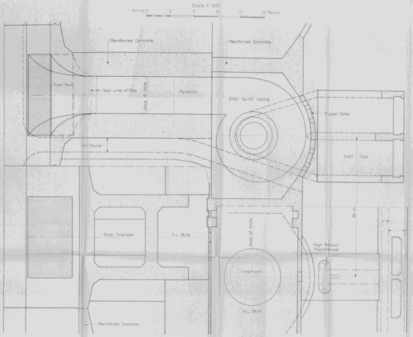
THE FOREBAY SCHEME

BUTTRESS INTAKE DAM

HORIZONTAL SECTIONS THROUGH POWER PLANT

HORIZONTAL SECTION THROUGH PENSTOCKS

HORIZONTAL SECTION AT R.L. 101



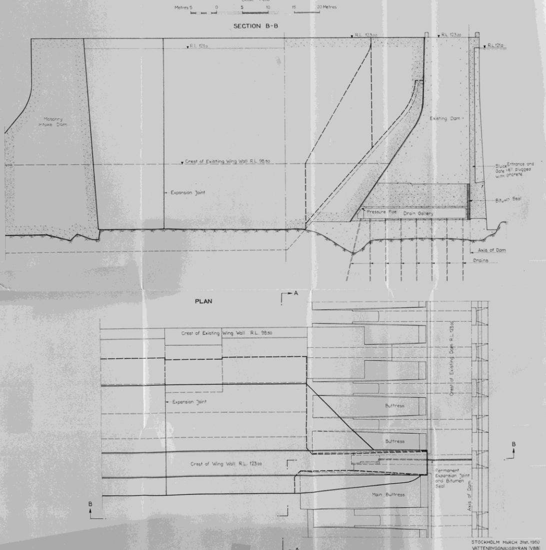
STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

THE FOREBAY SCHEME

EAST WING WALL PLAN AND SECTION

Scale 1:200



STOCKHOLM MARCH 31st. 1950
VATTENBYGGNADSBYRAN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

PLATE 8

THE FOREBAY SCHEME

EAST WING WALL SECTIONS AND DETAILS

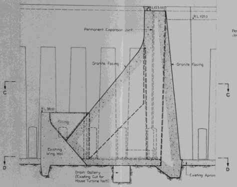
Scale 1:200 and 1:20

Scale

Metres 5 0 5 10 15 20 Metres

SECTION A-A

Scale 1:200



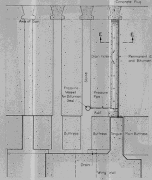
SECTION E-E

Scale 1:200



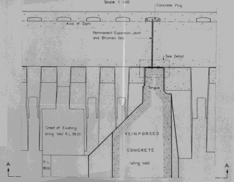
SECTION D-D

Scale 1:200



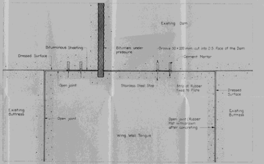
SECTION C-C

Scale 1:200



DETAIL OF SECTION C-C

Scale 1:20



STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

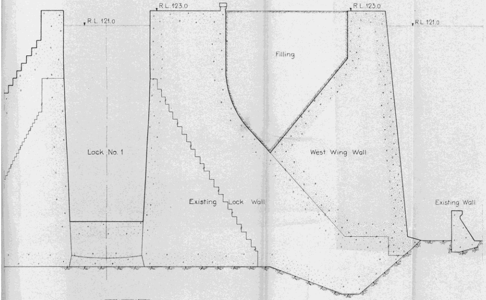
THE FOREBAY SCHEME

LOCK No. 1 AND WEST WING WALL

Scale 1:400 and 1:200

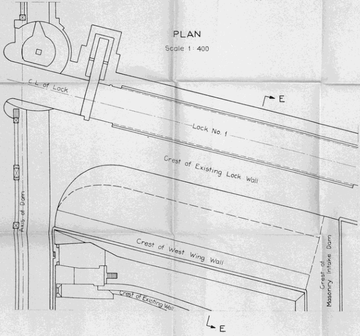
SECTION E-E

Scale 1:200

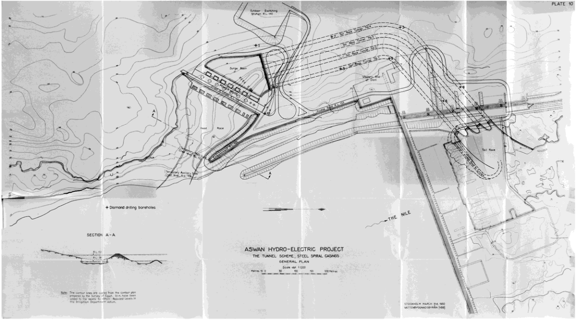


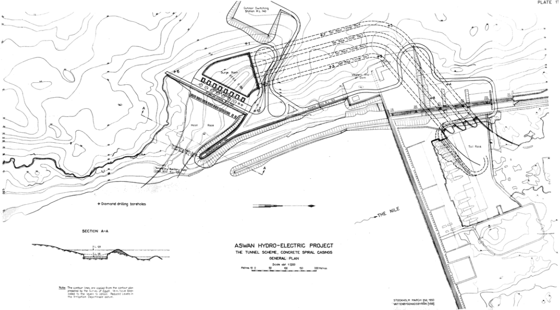
PLAN

Scale 1:400



STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)


Outdoor Switching Station R.L. 140
Surge Basin
Western End of Dam
Head Race
Tail Race
THE NILE
+ Diamond drilling boreholes
SECTION A-A
ASWAN HYDRO-ELECTRIC PROJECT
THE TUNNEL SCHEME, STEEL SPIRAL CASINGS
GENERAL PLAN
Scale abt 1:2000
Metres 10 0 50 100 150 200 Metres
Note: The contour lines are copied from the contour plan prepared by the Survey of Egypt. 15 m. have been added to the levels to obtain Reduced Levels in the Irrigation Department datum.
STOCKHOLM MARCH 31st 1930
VATTENBYGGNADSBYRÅN (VBB)


Outdoor Switching Station R.L. 140
Surge Basin
Western End of Dam
Head Race
Tail Race
+ Diamond drilling boreholes
THE NILE
SECTION A-A
ASWAN HYDRO-ELECTRIC PROJECT
THE TUNNEL SCHEME, CONCRETE SPIRAL CASINGS
GENERAL PLAN
Scale abt. 1:2000
Metres 10 0 50 100 150 200 Metres
Note: The contour lines are copied from the contour plan prepared by the Survey of Egypt. 1.6 m. have been added to the levels to obtain Reduced Levels in the Irrigation Department datum.
STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

THE TUNNEL SCHEME, CONCRETE SPIRAL CASINGS

LONGITUDINAL SECTION

Scale 1: 2000

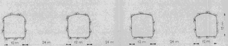
Scale

Metres 50 0 50 100 150 200 Metres



CROSS SECTION THROUGH TUNNELS

Scale 1: 500



STOCKHOLM MARCH 31st 1950
VATTENBYGGNADSBYRÅN (VBB)

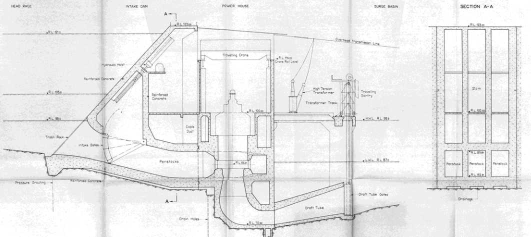
ASWAN HYDRO-ELECTRIC PROJECT

THE TUNNEL SCHEME, CONCRETE SPIRAL CASINGS

INTAKE AND POWER HOUSE

Scale 1:200

Metres 5 0 5 10 15 20 Metres



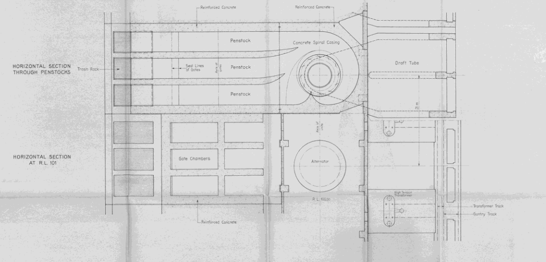
STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

THE TUNNEL SCHEME, CONCRETE SPIRAL CASINGS

HORIZONTAL SECTIONS THROUGH POWER PLANT

Scale 1:200



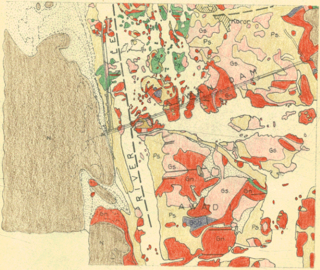
STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO-ELECTRIC PROJECT

ABSTRACT OF GEOLOGICAL MAP

OF THE FIRST ASWAN CATARACT 1907

BY JOHN BALL, Ph.D., B.Sc. F.G.S.



INDEX TO COLOURS

	Blown Sand	Gs.	Fine-grained Granite and Gneiss (including some Schists at South end of map)
Ps.	Alluvial Deposits	Sy.	Syenite
N.	Nubian Sandstone	Sch.	Micaceous and Hornblendic Schists
Gn.	Coarse-grained Granite	— — — —	Faults

Scale 1 : 20,000

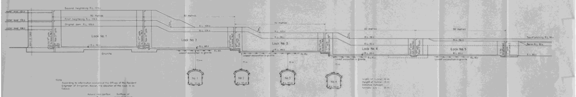
Metres 200 100 0 500 1000 1500 2000 Metres

ASWAN HYDRO-ELECTRIC PROJECT

LONGITUDINAL SECTION THROUGH LOCKS

(EXISTING STRUCTURE COPIED FROM DRAWING RECEIVED FROM THE IRRIGATION DEPARTMENT, ASWAN)

Scale 1:500



Note

According to information available at the Offices of the Resident Engineer of Irrigation, Aswan, the elevation of the rock is as follows:

	Natural rock surface R.L.	Surface of excavated rock R.L.
Lock No. 1	[illegible]	[illegible]
Lock No. 2	[illegible]	[illegible]
Lock No. 3	96	83
Lock No. 4	90 - 95	83
Lock No. 5	87 - 92	83
Bottom of Traffic Canal	[illegible]	[illegible]

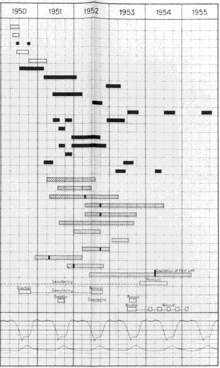
STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

ASWAN HYDRO — ELECTRIC PROJECT

THE TUNNEL SCHEME, CONCRETE SPIRAL CASINGS

CONSTRUCTION PROGRAMME

Item	Unit	Quantity
Preparation of contract documents		
Diamond rock drilling, Survey		
Placing of contracts		
Temporary access road for excavation		
Preliminary site works		
Excavation for intakes and power house and for parts of surge basin and head race	m^3	690,000
" of surge basin, main portion	m^3	430,000
" " " " , transport through tunnels	m^3	50,000
" " head race in the dry behind main sudd	m^3	240,000
" " " " " " " " auxiliary sudd	m^3	80,000
" " " " under water	m^3	40,000
" " tennel I and II, main portion	m^3	160,000
" " " I " II, portion beneath locks	m^3	15,000
" " " I " less than 80 m from foundation of dam	m^3	30,000
" " " III " IV, main portion	m^3	220,000
" " " III " IV, portion beneath locks	m^3	15,000
" " tail race in the dry	m^3	150,000
" " " " under water	m^3	20,000
Concreting of intakes	m^3	90,000
" " power house, assembly bays		72,500 (bracketed, four rows)
" " " " , substructure including spiral casings		
" " " " , superstructure		
" " " " , substructure stage II		
" in tunnels	m^3	15,000
Erection of intake gates		
" " draft tube gates and cranes		
" " travelling cranes		
" " steel sheet lining of draft tubes		
" " stay rings		
" " turbo - alternators		
Temporary sudd for tail race		
" " " intakes and power house (main sudd)	$\Sigma\ bh^2$	26,000
" " " head race (auxiliary sudd)	$\Sigma\ bh^2$	4,000
" stop logs for draft tubes		
Upstream water level		
Downstream water level (approximate level in western channel)		



This construction programme can be followed only if large capacity equipment for excavation and transport of rock is provided at the site

The following machines may be used

5 Excavators with shovel equipment for 3.5 cuy and dragline equipment for 5 cuy
2 " " " " " 1.5 cuy
20 Trucks for 20 tons load
10 " " 13 tons "

To day this equipment may be purchased for delivery in New York within 4 to 6 weeks

Key:

- Rock excavation
- Placing of concrete
- Erection of machinery etc.
- Miscellaneous
- Date of completion of work for the first unit to be put into operation

STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)

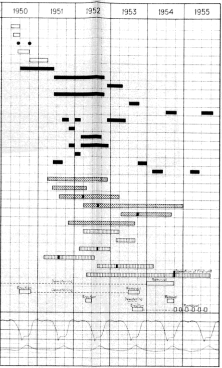
EN

ASWAN HYDRO – ELECTRIC PROJECT

THE TUNNEL SCHEME, STEEL SPIRAL CASINGS

CONSTRUCTION PROGRAMME

Item	Unit	Quantity
Preparation of contract documents		
Diamond rock drilling, Survey		
Placing of contracts		
Temporary access road for excavation		
Preliminary site works		
Excavation for intakes and power house and for parts of surge basin and head race	m^3	820,000
" of surge basin, main portion	m^3	480,000
" " " " , transport through tunnels	m^3	50,000
" " head race in the dry behind main sudd	m^3	370,000
" " " " " " " " auxiliary sudd	m^3	100,000
" " " " under water	m^3	40,000
" " tunnel I and II, main portion	m^3	110,000
" " " I " II, portion beneath locks	m^3	15,000
" " " I " less than 80 m from foundation of dam	m^3	30,000
" " " III " IV, main portion	m^3	220,000
" " " III " IV, portion beneath locks	m^3	15,000
" " tail race in the dry	m^3	150,000
" " " " under water	m^3	20,000
Concreting of intakes	m^3	160,000
" " power house, assembly bays		55,000 (rows: assembly bays, substructure stage I, superstructure, substructure stage II)
" " " " , substructure stage I		
" " " " , superstructure		
" " " " , substructure stage II		
" in tunnels	m^3	15,000
Erection of intake gates		
" " draft tube gates and cranes		
" " travelling cranes		
" " steel sheet lining of draft tubes		
" " steel spiral casings		
" " turbo – alternators		
Temporary sudd for tail race		
" " " intakes and power house (main sudd)	Σbh^2	26,000
" " " head race (auxiliary sudd)	Σbh^2	4,000
" stop logs for draft tubes		
Upstream water level		
Downstream water level (approximate level in western channel)		



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STOCKHOLM MARCH 31st, 1950
VATTENBYGGNADSBYRÅN (VBB)